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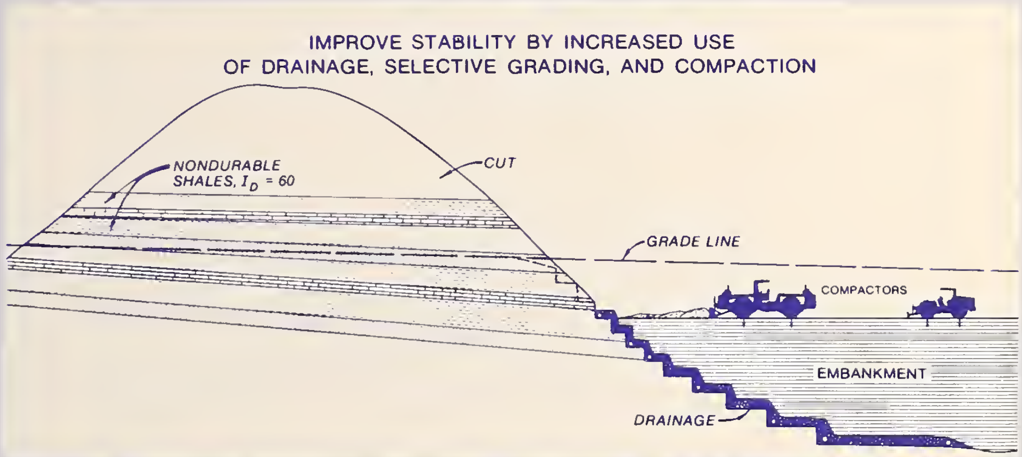
ESIGN AND CONSTRUCTION OF COMPACTED SHALE EMBANKMENTS

Vol. 5. Technical Guidelines



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Final Report

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Prepared for

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FEDERAL HIGHWAY ADMINISTRATION

Offices of Research & Development

Washington, D. C. 20590

FOREWORD

This report provides technical guidance on all aspects of the design and construction of compacted shale embankments. Also covered are evaluation techniques for existing distressed shale embankments and remedial treatment methods that can be used as correction schemes. The report will be of interest to chief highway administrators and other policymakers, as well as to engineers at the operating level.

The technical guidelines presented in this report resulted from FCP Project 4D study, "Development of Methodology for Design and Construction of Compacted Shale Embankments," conducted by the U.S. Army Engineer Waterways Experiment Station (WES). The research was performed by the WES for FHWA under Purchase Order No. 4-1-0196 during the period July 1, 1974 to January 30, 1979.

Acknowledgement is given to the State Transportation Agencies who are cited by the authors in the preface of this report. Special thanks are extended to the following State highway personnel who served on an advisory group that provided consultation to the research program: Mr. Joseph E. Armstrong (Montana), Mr. Roland E. Bashore (Ohio), Mr. Henry A. Mathis (Kentucky), Mr. George S. Meadors, Jr. (Virginia), Mr. Rod H. Prysock (California), Mr. David L. Royster (Tennessee), Mr. William J. Sisiliano (Indiana), and Mr. Berke L. Thompson (West Virginia). Special thanks are also extended to Dr. James K. Mitchell of the University of California, Berkeley, and to Dr. Leonard E. Wood of Purdue University.

Although shale is the most common (and the most troublesome) type of rock in the United States, it is not a nationwide problem. Accordingly, sufficient copies of the report are being distributed by FHWA Bulletin to provide a minimum of one copy to each FHWA Regional Office, one copy to each FHWA Division Office, and one copy to each State highway agency. Additional copies are being sent to those geographical areas that are significantly affected by troublesome shales which exhibit a wide spectrum of engineering behavior.

Charles F. Scheffey
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Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

This fifth report provides guidance on geological investigations, durability classification of shales, design features, and construction procedures unique to compacted shale embankments for highways. Guidance is also given on techniques for evaluating existing shale embankments and remedial treatment methods for distressed shale embankments. Index tests and classification criteria for determining shale durability, techniques for evaluating excavation characteristics, and alternative procedures for excavation, placement and compaction of shales to achieve adequate stability and minimum settlement are described. The use of drainage measures, selective excavation and placement of nondurable shales in thin lifts with procedural compaction provisions based on field test pads is emphasized.

This final report is the fifth in a series. The other reports are:

Volume	FHWA No.	Title
1	RD-75-61	Survey of Problem Areas and Current Practices
2	RD-75-62	Evaluation and Remedial Treatment of Shale Embankments
3	RD-77-1	Slaking Indexes for Design
4	RD-78-140	Field and Laboratory Investigations, Phase III

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PREFACE

The study of the methodology for design and construction of compacted shale embankments is a 4-year investigation funded by the U. S. Department of Transportation, Federal Highway Administration, under Intra-Government Purchase Order No. 4-1-0196, Work Unit No. FCP 34D5-012.

The work was initiated during June 1974 by the Geotechnical Laboratory (GL) of the U. S. Army Engineer Waterways Experiment Station (WES). Mr. William E. Strohm, Jr., Research Group, Engineering Geology and Rock Mechanics Division (EGRMD), GL, was the principal investigator during the period of this report. The work reported herein was performed under the supervision of Mr. Strohm and by Messrs. George H. Bragg, Jr., and Timothy W. Zeigler, formerly of EGRMD. The report was prepared by Messrs. Strohm, Bragg (Part VIII), and Zeigler (Part IX). The investigation was accomplished under the general supervision of Dr. Don C. Banks, Chief, EGRMD, and Mr. James P. Sale, Chief, GL.

Personnel of the State highway agencies in California, Colorado, Indiana, Oklahoma, Ohio, Oregon, Kentucky, Missouri, Montana, North Carolina, North Dakota, New York, Pennsylvania, Tennessee, Utah, Virginia, and West Virginia provided information for the study and assisted in obtaining shale samples.

Directors of WES during the conduct of this portion of the study and preparation of the report were COL G. H. Hilt, CE, and COL John L. Cannon, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
mils	0.0254	millimetres
gallons (U. S. liquid)	3.785412	cubic decimetres
pounds (mass)	0.4535924	kilograms
tons (2000 lb, mass)	907.1847	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	6894.757	pascals
pounds (force) per square foot	47.88026	pascals
tons (force) per square foot	95.76052	kilopascals
degrees (angle)	0.01745329	radians

DESIGN AND CONSTRUCTION OF COMPACTED SHALE EMBANKMENTS; VOL. 5, TECHNICAL GUIDELINES

PART I: INTRODUCTION

Background

1. Construction of the modern highway system in much of the United States has required large, high embankments using economically available shale* from adjacent cuts or borrow areas. Settlements of 1 to 3 ft in many shale embankments have required frequent overlaying to maintain grade. Raising of bridge abutments founded on approach embankments of shale has also been required. In some shale embankments, continuing settlements are followed by slope failure and slides; while in others the settlement stops and no further distress occurs. The more severe settlements and slope failures have occurred in the East Central States where the climate is humid. Repair of failures is expensive, amounting to nearly $2 million in one case for three slides where reconstruction was required over a period of 18 months.

2. The underlying cause of excessive settlement and slope failures in highway shale embankments appears to be deterioration or softening of certain shales with time after construction. Some shales are rock-like when excavated, but when placed as rockfill deteriorate or soften into weak soil. Other shales, often interbedded with limestone or sandstone, break down when excavated, but large-size durable rocks often prevent adequate compaction. The difficulties encountered in using shale in highway embankments are complicated by variations in geology and physical properties of sedimentary rocks, depth of weathering, climate and groundwater conditions, and the weather and construction methods. *The main difficulty is determining which shales can be placed as rockfill in thick lifts (2 to 3 ft) and which shales must be placed as soil and compacted in thin lifts (8 to 12 in.).*

3. The need for comprehensive guidance on the use of shales in highway embankments and procedures for evaluation and treatment of existing shale embankments led to a 4-year study and the development of this report of technical guidelines. The following reports** were published during the study:

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* Shale is used as a general term for weak fine-grained sedimentary rocks such as claystone, siltstone, mudstone, etc.

** Referenced by volume number in this report.

a. "Design and Construction of Compacted Shale Embankments, Vol. 1, Survey of Problem Areas and Current Practices," Interim Report, August 1975, Report No. FHWA-RD-75-61.

b. "Design and Construction of Compacted Shale Embankments, Vol. 2, Evaluation and Remedial Treatment of Shale Embankments," Interim Report, August 1975, Report No. FHWA-RD-75-62.

c. "Design and Construction of Compacted Shale Embankments, Vol. 3, Slaking Indexes for Design," Interim Report, February 1977, Report No. FHWA-RD-77-1.

d. "Design and Construction of Compacted Shale Embankments, Vol. 4, Field and Laboratory Investigations, Phase III," Interim Report, October 1978, Report No. FHWA-RD-78-140.

Purpose

4. This report is intended to provide technical guidelines for use by State Highway geotechnical and construction engineers. Portions of the report have been prepared to facilitate desired extraction by the State geotechnical engineer for use in instructions in field personnel at the State Highway District level.

Scope

5. This report covers field exploration and sampling of shales, laboratory testing and classification, design features for shale embankments, construction methods and control procedures, evaluation methods for existing shale embankments, and remedial treatment methods for distressed shale embankments. The scope is limited to methods and procedures needed for shale embankments that differ from those normally used for soil or hard rock embankments. The material covered excludes foundations, cut slopes, and frost action.

PART II: GENERAL CONSIDERATIONS FOR SHALE EMBANKMENTS

Basic Concepts

6. *The successful use of excavated materials from cuts in shale formations for highway embankments requires adequate compaction of all fill materials and sufficient drainage to prevent harmful saturation of the completed embankment.* These two main requirements are often difficult to achieve because of the variable stratification of shale formations. Features of shale formations in cuts and other borrow sources, as indicated in Table 1, have an important influence on the type and extent of design measures (foundation benching, drainage provisions, material usage, compaction requirements, and embankment slopes) and special excavation, placement, and compaction procedures required during construction.

7. Shale formation features in cuts and other borrow areas should be considered early in the preliminary design to assess the need for specifying and the feasibility of controlling selective excavation and separate placement and compaction of (a) durable shale and rock in rockfill lifts (at the base of the embankment and/or outer shells of the embankment) and (b) nondurable shale and soil in thin lifts (or inner sections of embankments). As an alternative, the cost of breaking down all materials during excavation and placement for compaction in thin lifts should be compared with selective excavation and placement to arrive at the best solution. This comparison may be required on a cut-by-cut basis for projects in complex formations. In highly variable formations, unclassified excavation may be justified as discussed in paragraph 9. The cut sections shown in Figure 1 illustrate some of the difficult stratigraphic and shale conditions that require special construction procedures to achieve adequate compaction and drainage:

<u>a</u>. Placement of nondurable, hard shale interbedded with limestone (Figure 1a) as rockfill in thick lifts could result in large settlements and possible slope failure several years after construction. *In a humid climate, where surface drainage and groundwater seepage could infiltrate the embankment, the shale could soften and deteriorate into clay.* Two possible solutions are noted in Figure 1a. The feasibility either of breaking down and compacting all material as soil in thin lifts or of using selective grading to separate out the limestone would depend on the cost of the required excavation technique. If blasting were necessary and the limestone layers were thin (3 to 5 ft), selective excavation would probably be impractical, and the most economical solution might be extra blasting (away from cut slope line) to break down all materials for compaction, as soil.

Table 1. Influence of Shale Formation Features in Cuts and Other Borrow Sources and Construction of Shale Embankments

Feature	Influence
1. Thickness of soil and weathered materials in cuts and depth to unweathered materials in fill areas.	Thick soil and weathered shale/rock cover in cuts may require stockpiling or selective excavation and placement in the upper portion of other embankments, especially on sidehill locations. Thick soil and weathered shale/rock cover on sidehill fill areas requires considerable excavation and stockpiling or selective placement in other embankments (being completed) to obtain required underbenching into unweathered shale/rock, depending on groundwater seepage conditions.
2. Inclination (strike and dip), thickness and spacing of shale layers and interbedded harder rock (such as sandstone and limestone).	Cut sections of stratified shales of different hardness and durability and harder rock strata may require extra blasting and/or heavy compaction equipment to break down all materials for compaction in thin lifts or selective excavation and placement to compact nondurable shales in thin lifts and durable shales and rock as rockfill in base drainage layers and/or outer embankment sections.
3. Hardness and durability of shales in different layers and hardness of limestone/sandstone layers.	Influences required excavation effort (ripping and/or blasting) to meet placement and compaction requirements.
4. Frequency and spacing of joints and bedding planes.	Influences maximum size and shape (blocky or slabby) of shale and harder rock pieces produced by ripping or blasting in cuts.
5. Groundwater conditions; springs, seepage patterns, and seasonal variations.	Influences type and extent of underdrains and other drainage measures required on cut-fill transition slopes and benched sidehill slopes, considering quantity of rockfill available for foundation surface drainage layers.

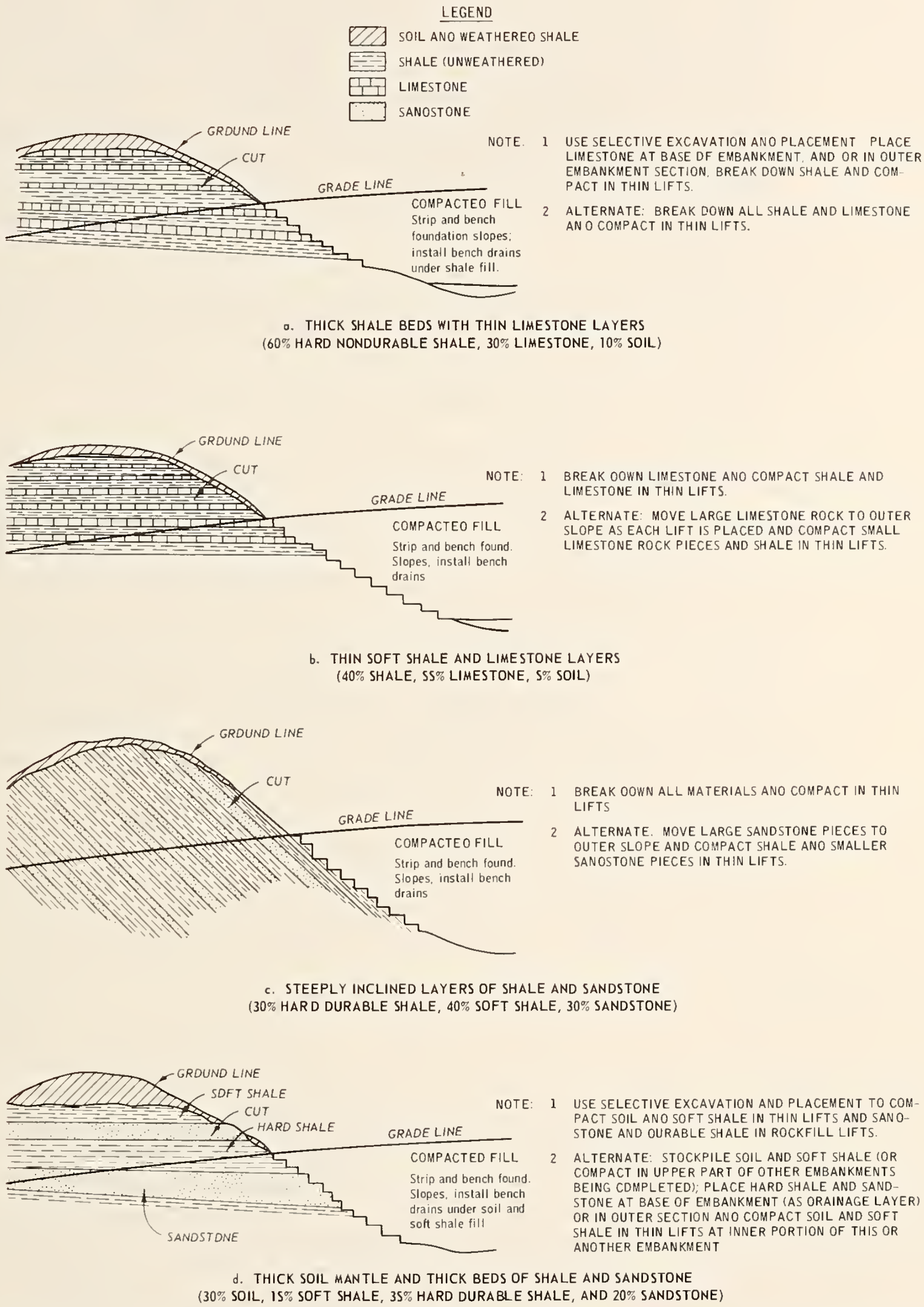


Figure 1. Examples of difficult shale and stratigraphic features and possible solutions

b. A cut section containing alternate thin beds (6 in. to 2 ft) of soft shale and limestone (Figure 1b), which could be excavated by ripping, would require placement as soil. Large limestone slabs would have to be broken down or pushed to the outer slope. *Although 55 percent of the cut is limestone, placement of the material as rockfill should not be considered since the percentage of limestone could vary from 0 to 100 percent in different scraper or hauler loads.* Selective excavation to segregate shale and limestone for placement in separate parts of a lift (shale in the central part and limestone in the outer part) may not be feasible.

c. Steeply inclined beds of different shales and harder rocks such as sandstone (Figure 1c) would be very difficult to segregate. The best recourse might be to break down all materials and place them as soil in thin lifts.

d. A cut section with a thick (10 ft or more) soil mantle and thick shale and harder rock strata (Figure 1d) can be selectively excavated and placed in separate lifts. Use of selective excavation and grading over several cuts and fills would allow soil and soft shales to be compacted in thin lifts in the upper portion of embankments nearing completion. The hard durable shale and sandstone should then be placed as rockfill at the base of embankments being started. *The stability of hard durable shale and sandstone placed as rockfill requires a large number of contacts between individual rocks.* These contacts cannot be achieved unless the amount of fines (soil and gravel sizes) is less than about 20 to 30 percent of the total material. Placing more than 20 to 30 percent soil and soft shale in the same lift with durable shale and/or rock produces loose fines around the durable rock. The loose fines can cause large settlements upon wetting and eventual slope failures.

8. The above examples emphasize the importance of recognizing when special construction procedures are required to obtain proper placement of shales and harder rock before adequate compaction can be accomplished. Wherever possible durable rock should be used at the base of shale embankments to provide good drainage and prevent possible build up of pore water pressures that could lead to instability. The need for and selection of special construction procedures depends on the amount of detailed information developed during field and laboratory investigations concerning (a) geologic and groundwater conditions, (b) hardness and durability of shale strata, and (c) variations of these features within designated cut sections along the project length. Without detailed information, a conservative approach of requiring extensive drainage measures and requiring all materials to be broken down and compacted as soil would increase the construction costs. *The extra cost might be unwarranted.*

9. Unclassified excavation and placement is another alternative to special excavation (to break down all materials) and selective excavation and placement. This alternative may be necessary for highly variable formations in which complete sampling and testing of various strata would be prohibited by the excessive cost. The full-time presence of experienced inspectors is required not only to inspect but also to classify excavated shale materials and designate which materials require compaction as soil in thin lifts and which materials can be placed as rockfill with or without special compaction. The advantages and disadvantages of the three methods are compared in Table 2. Requirements for accomplishing these methods that should be considered during the design are shown in Figure 2. The need for special construction procedures is a primary consideration in developing a rational evaluation and design process (outlined in Figure 3) for construction of new shale embankments.

10. During corridor studies, the geotechnical engineer's knowledge and experience should be fully used. Shale formation features and durability index tests on selected core samples of shales, together with a review of recent design, construction, and performance experience, lead to a logical evaluation of potential problems. This evaluation is a basic step in determining requirements for detailed field investigations in the preliminary design stage and the need for specifying additional breaking down of shales and rock during excavation and placement. Once this need has been decided, the applicability of laboratory testing and the quantity, type, and size of shale samples as well as sample preparation techniques and testing procedures during final design can be determined. The need for test pads during construction and for procedural-type compaction specifications also can be evaluated at this stage. The process shown in Figure 3 provides a logical method for selecting design parameters for both short-term and long-term performance evaluations, proper design features, and required construction procedures for shale embankments. Short-term performance refers to the first several years after construction when large deformations can occur due to compression of loose materials upon wetting, but decrease in strength due to softening is small. Long-term refers to large possible decreases in strength due to saturation and softening of shale 3 to 10 years after construction.

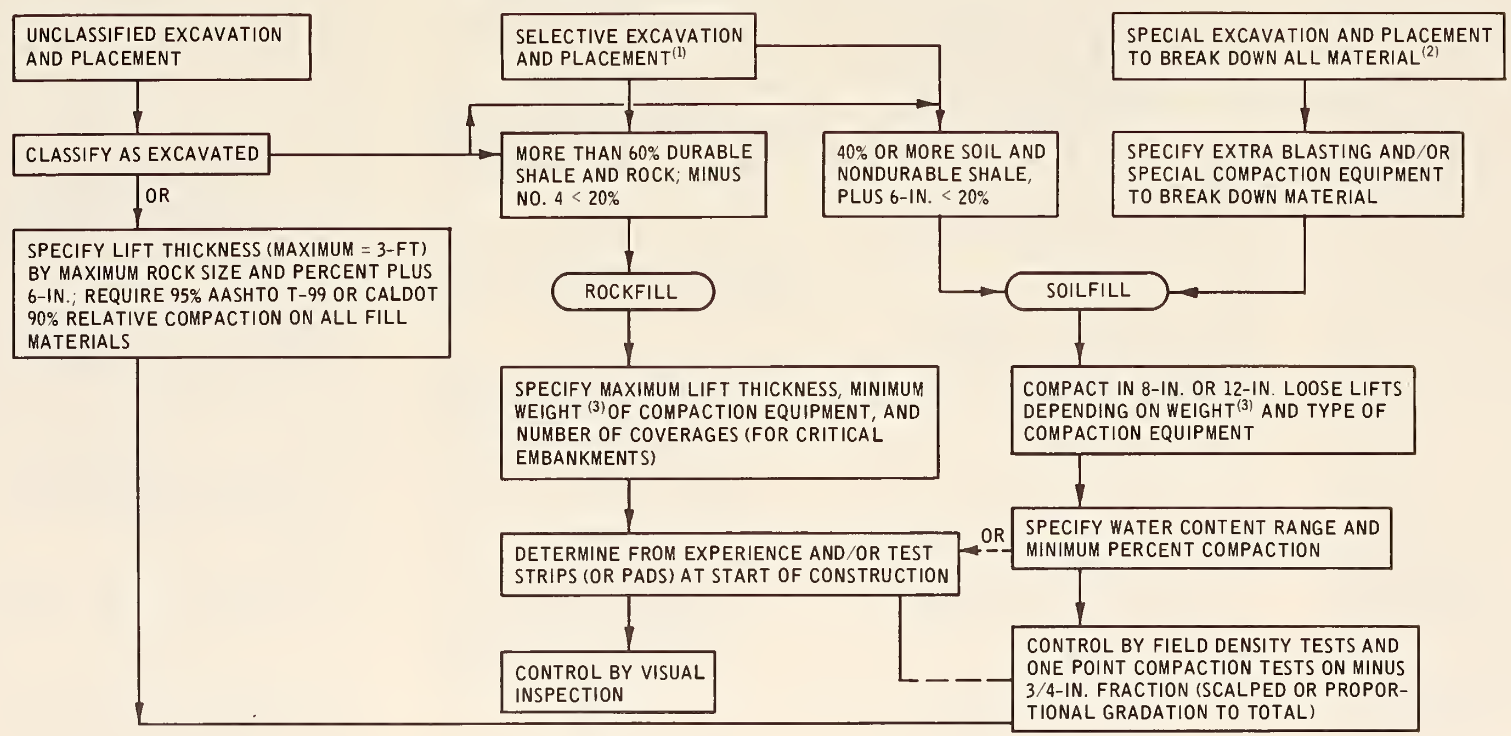
Field Exploration and Sampling

11. The basic objective of field exploration and sampling for shale embankments is to define the formation features of each cut and other borrow areas and to obtain samples of different shale layers for durability index tests, and natural water content, compaction, and special compression and strength tests. *At least two core borings in addition to the usual auger borings are required in each cut or borrow area to define soil and weathered shale depths and the thickness and inclination of different strata.* The coring of shales and harder rock layers should

Table 2. Advantages and Disadvantages of Construction Methods for Shale Embankments

Method	Advantages	Disadvantages
Special Excavation and Placement to Break Down All Materials and Compact as Soil	Simplifies geotechnical work during design. Simplifies construction procedures. Insures better compaction and stability. Increases potential for less maintenance and better performance after construction.	Increases construction costs. Uneconomical material use. Increases required underdrainage. Requires good project inspection to enforce.
Selective Excavation and Placement to Compact Nondurable Shales as Soil and Durable Shale and Rock as Rock Fill	Enables proper use of materials and successful use of shales. Leads to rational procedures for field investigations, testing and selection of design features. Provides detailed geologic profiles and sections. Reduces unknowns and increases potential for less maintenance and better performance after construction.	Requires more detailed field investigations and index testing of shales. Requires geologic profiles and sections showing fill type and placement location in embankment on sections. May require stockpiling or complex excavation and placement sequence. Requires good project inspection to enforce.
Unclassified Excavation and Placement	Requires least amount of geotechnical investigation during design. Requires less information on construction plans and fewer special provisions.	May cause uneconomical use of materials. Increases chances for undesirable mixing of rock with nondurable shale and soil. Requires full-time experienced fill inspectors on each segment of project to classify excavated materials. Could increase construction costs to include unknown contingencies. May require conservative design assumptions; more drainage measures, flatter slopes, and other measures.

CONSTRUCTION ALTERNATIVES FOR SHALE EMBANKMENTS



NOTE: (1)DESIGNATE FILL TYPE BY DEPTH ZONES ON GEOLOGIC SECTIONS AND/OR PROFILES OF EXCAVATION AREAS AS PART OF CONSTRUCTION PLANS. PLACEMENT LOCATION SPECIFIED ON EMBANKMENT SECTIONS. ENFORCED BY FIELD INSPECTIONS.

(2)DESIGNATE BY STATION OR ON GEOLOGIC SECTIONS IN CONSTRUCTION PLANS. ENFORCE BY FIELD INSPECTION.

(3)MINIMUM 30 TON STATIC WEIGHT.

Figure 2. Construction alternatives for shale embankment

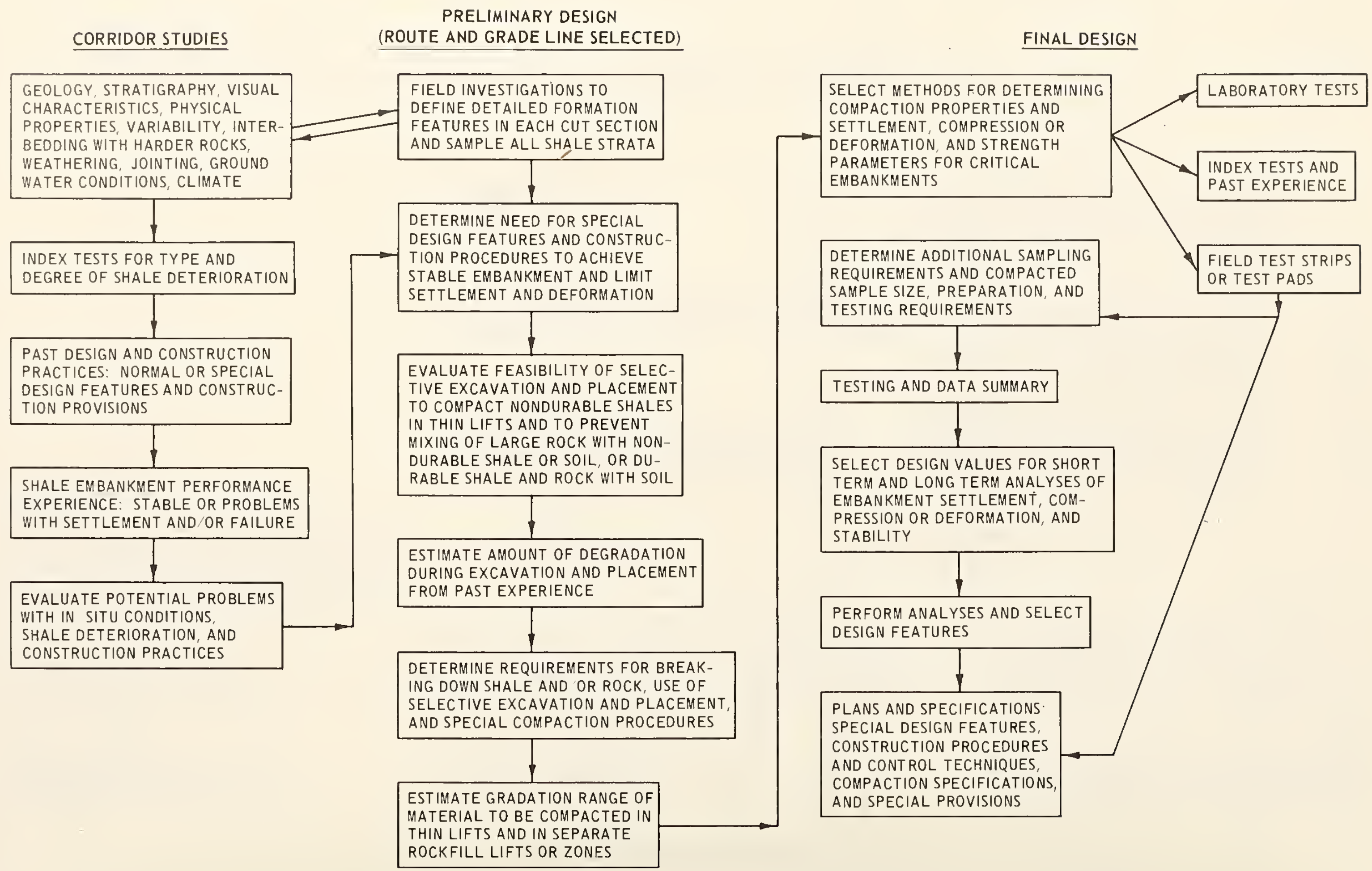


Figure 3. Geotechnical evaluation and design process for new construction of shale embankments

be extended to a sufficient depth to detect layers draining into shale embankment areas at the fill-cut transition. Measurement of groundwater elevations in core borings can be used to define subsurface seepage that would enter the embankment. Aerial photographs (stereo black and white, color, and color infrared) and thermal infrared imagery provide valuable information on geological conditions, surface drainage channels, exit patterns of subsurface seepage, and springs. Examples and recommended procedures on field investigations and sampling are presented in Part III.

Durability Classification of Shales

12. *The most important step in the design of shale embankments is the classification of shales according to their long-term durability (i.e., susceptibility to deterioration).* The slake durability index, I_D , and jar soaking index, I_J , are two simple aids for defining deterioration (but not hardness). The tests can be used as a field identification aid during construction, when supplemented by a rapid drying technique (i.e., microwave oven). The point-load tests can also be used as an expedient field index test during construction to identify shales susceptible to deterioration provided that the point-load index can be correlated with slake-durability index during the geotechnical investigation. Shales classified as mechanically hard and durable (e.g., $I_D > 90$, $I_J = 6$) can be used as rockfill (without excessive fine-grained material, paragraph 7d); while shales classified as soft nondurable (e.g., $I_D < 60$, $I_J \leq 2$) need to be compacted as soil in thin lifts. However, intermediate shales classified as hard nondurable (e.g., $I_D = 60$ to 90 , $I_J = 3$ to 5) are difficult to distinguish and require special treatment (e.g., a high degree of compaction and isolation from infiltrating water to prevent wetting). Shale classification criteria are described in Part IV.

Shale Formation Excavation Characteristics

13. Defining the excavation characteristics of shale formations is an important requirement for shale embankments. The in situ hardness of nondurable shales and the amount of interbedding with harder rocks control the excavation methods required to obtain the breakdown necessary for adequate compaction in thin lifts. The breakdown during excavation depends on the amount of ripping or blasting. During placement, further breakdown depends on the weight and type of compaction equipment. *For example, the use of heavy (30-ton) tamping rollers that can break down shale and limestone during compaction would require less breakdown by extra blasting during excavation.*

14. Classification of excavation characteristics of shale formations can be based on seismic velocity or on locally developed special

descriptive designations and index numbers indicating the degree of excavation difficulty. Information on the type of material, stratigraphy, and jointing is required to arrive at a final classification and required excavation procedures. Excavation criteria are described in Part V.

Geotechnical Design Features for Shale Embankments

15. The main considerations to assure satisfactory performance of shale embankments include foundation benching, drainage provisions, material usage, compaction requirements, and slope inclination, as discussed briefly below.

a. Benching and drainage. On natural slopes of weak materials, such as weathered shale or colluvium, benching into unweathered material and permeable rock drainage blankets, underdrains, or bench drains should be used to prevent saturation of embankment shales. Horizontal drains also may be required to intercept and lower excessive groundwater seepage in jointed limestone or sandstone layers. Surface drainage features include curbing, gutters and lined ditches.

b. Material usage. A material usage plan and construction excavation requirements should be developed for the project so that durable shales and rock are placed as rockfill, and nondurable shales and soil are compacted as soil in thin lifts. The alternative is to require that all excavated materials be broken down and compacted as soil. Excavation and compaction requirements should be tailored to meet gradation criteria (Figure 2) for adequate compaction of nondurable shales into dense, relatively impervious layers and placement of durable shales and rock into relatively free draining rockfill layers with good contacts between large rocks. Special provisions for shale test pads during construction may be necessary to develop required compaction methods and control procedures for nondurable shales. *Compaction in excess of 95 percent AASHTO T-99 maximum density may be required for coarse-graded, nondurable shales, since these shales often develop large strains before reaching adequate shear strengths for stability.* The resulting compression (which increases upon wetting) can cause large settlements.

c. Slopes. Shale embankment slopes usually should not be steeper than 2:1, unless a significant portion of the outer embankment section is durable rockfill or use of steeper slopes of soillike shales is verified by stability analyses. Flatter slopes of 3:1 in addition to good compaction may be required for bridge approach fills (to reduce lateral deformation) where settlements must be limited. Flatter slopes may be a logical alternative when other design measures are impractical or when construction might not be well controlled.

d. Special design features. Special design features for problem areas such as unstable hillside locations, narrow right-of-way areas where high embankments require steep slopes, or areas of excessive seepage include (a) berms, shear trenches with underdrains, retaining structures (rock buttresses, reinforced earthwalls, gabion walls, crib walls, etc.) and (b) spring drains, longitudinal and lateral underdrain or rock drainage blankets (or pads), and horizontal drains. Royster (1975)* describes the application of special design features for sidehill embankments. Recommended applications, design criteria, and examples are described in Part VI and IX.

Construction of Shale Embankments

16. Construction of shale embankments, described in Part VII, requires proper execution and inspection of the following items:

a. Foundation preparation. Benching of foundation slopes, installation of bench drains, underdrains, and horizontal drains to intercept subsurface seepage from springs and permeable strata covered by the embankment.

b. Excavation procedures. Breaking down or selective excavation of soil, nondurable shale, durable shale, and rock.

c. Construction sequence. Selective placement and/or stockpiling to use soil and weathered materials from cut sections and/or foundation benches in the upper portion of embankments being completed; selective placement of rock and hard, durable shale as a drainage layer on foundation slopes, and/or in outer embankment sections; and selective placement of nondurable shale as soil in thin lifts.

d. Compaction equipment capabilities. Requiring proper type and adequate weight of compaction equipment to further break down shales and compact maximum loose lift thickness

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* Royster, D. L., "Tackling Major Highway Landslides in the Tennessee Mountains," Civil Engineering, American Society of Civil Engineers, Sep 1975.

allowed by specifications and/or special provisions or determined by test pads at the start of construction.

e. Compaction procedures. Preventing undesirable mixing of excessive amounts of large rock with nondurable shale placed as soil; preventing the mixing of large amounts of soil and nondurable shale with rock placed in thick lifts; and requiring adequate compaction using the proper combination of equipment and number of coverages.

f. Compaction control. Checking material type; limiting quantity of rock in material placed as soil; limiting quantity of fines in rockfill; checking lift thickness, water content, density, and percent compaction (where appropriate) of shales placed as soil in thin lifts.

g. Construction of berms, rock buttresses, and special retaining structures (defined in Part IX). To ensure the use of proper materials and procedures:

(1) Berms. Adequate foundation benching and drainage.

(2) Buttresses. Use of hard, durable rock and adequate filter material along the base and behind the buttress to prevent erosion of fine-grained material into rock and clogging of drainage paths.

(3) Reinforced earth. Use of free-draining backfill material (clean sands or gravelly sand).

(4) Gabion walls. Clean stone for filling baskets and proper filter material behind wall to prevent erosion of fine-grained material into stone fill and clogging of drainage paths.

Evaluation of Shale Embankments

17. Determining the current stability and future performance of existing shale embankments is a major problem, especially in the East Central States where many large shale embankments are exhibiting distress. The distress, in the form of continuing settlement, cracking along pavement edges and shoulders, small shoulder slides, and slope sloughing, has often developed within 1 to 10 years, into a large slide requiring expensive reconstruction, especially along sidehill locations (Vol. 2). In many cases, the distress has been handled at the District level by maintenance forces without the assistance of the State geotechnical staff until a problem reaches major proportions. Several States are cataloging distressed shale embankments and assigning priorities according to the seriousness of distress and consequences of failure.

These states have established a continuing program of limited field investigations to evaluate the distressed embankments in order of priority and available funds.

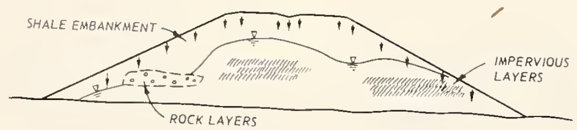
18. *The primary cause of shale embankment distress is saturation and progressive softening and deterioration of nondurable shales (often mixed with rock and soil) by surface water and/or subsurface seepage that enters the embankment during periods of prolonged rainfall.* The examples shown in Figure 4 illustrate sources of infiltrating water and resulting variations in saturation zones. Because of the heterogeneous mixture of shales, rock, and soil in many shale embankments (Vol. 4), the infiltrating water follows erratic flow paths, depending on the relative porosity of different layers and the pattern of cracks caused by settlement and deformation. Consequently, it is often difficult to define the pattern of infiltration, location of soft shale or soil zones, and the extent of distress without extensive subsurface investigations.

19. *The major objective in evaluating the future behavior of distressed shale embankments is to determine whether settlements will eventually stop or will continue and develop into a large slide.* The first step should be a thorough field evaluation by a geotechnical engineer to determine the surface extent and seriousness of the distress and the probable effectiveness of immediate remedial measures in reducing further distress. A first step that is usually inexpensive is improvement of surface drainage to reduce infiltration. Other steps in the evaluation process for major embankments include the following:

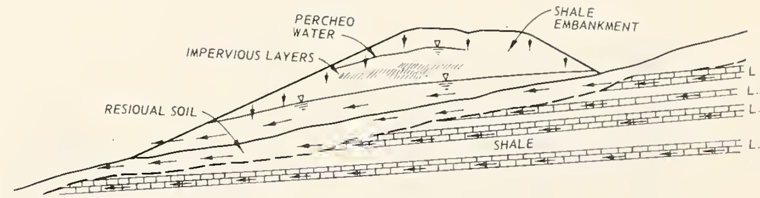
a. Review of available information on the stratification and attitude of bedding at the cut-fill transition and uphill of sidehill embankments, groundwater seepage patterns, placement sequence and amount of mixing of different materials during construction, quantity and durability of shales placed in the embankment, and past experience with similar embankments in the same formation and local area to establish possible seepage sources and locations of weak zones.

b. Periodic roadway center-line and cross-section elevation surveys to monitor the rate of settlement and lateral deformation with time.

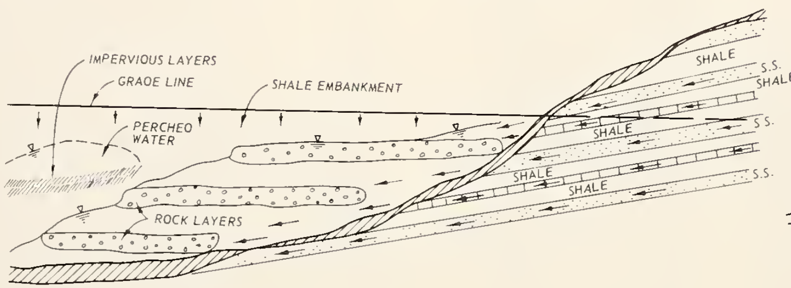
c. Disturbed sample borings to define the depth and thickness of predominant types of materials (i.e., shale, shale chunks and clay soil, soft shale with some limestone, limestone with few shale chunks, sandstone); type and amount of shale deterioration (e.g., softened into clay or fragmented into hard silty chips and gravel sizes, or friable clayey chunks); location of wet or saturated zones; and type of foundation material (e.g., sandstone drainage layer, thick weathered shale, hard shale, and limestone strata).



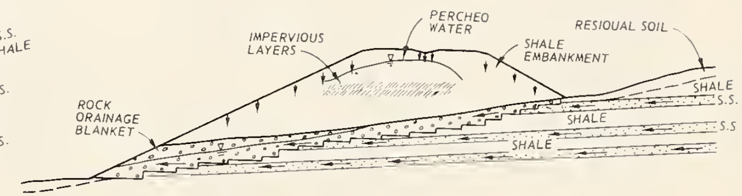
a. Symetrical (thru) embankment



c. Sidehill embankment; no benching or drainage blanket



b. Fill-cut transition



d. Sidehill embankment with benching and drainage blanket

Figure 4. Examples of infiltrating water sources and saturation zones in shale embankments

d. Installation of piezometers and/or slope inclinometer casing in selected borings to monitor groundwater levels (or pore water pressures from piezometers) and lateral movements and development of a slip zone.

e. Logging of selected borings with portable nuclear moisture density equipment to locate wet, low density zones.

f. In situ pressuremeter tests and/or undisturbed sampling and laboratory testing to estimate shear strengths for evaluation of stability.

The scope of the field investigation and monitoring program depends on the type of available information and experience in the area. The results of the evaluation should provide necessary information for decisions on the need for remedial measures and the selection and design of appropriate treatment methods. Guidance on evaluation techniques for shale embankments is summarized in Part VIII.

Remedial Treatment of Shale Embankments

20. The primary consideration in remedial treatment of shale embankments should be surface and subsurface drainage methods. Drainage methods are an integral part of most remedial treatment methods. Remedial treatment plans should include surface treatment and drains designed to minimize infiltration of surface water. Subsurface drainage is essential in treatment of sidehill and transitional fills. Certain types of subsurface drains can be rapidly installed (i.e., horizontal drains, pumped vertical wells) and are used when temporary (or emergency) support is required. Early installation of subsurface drains, when feasible, can halt embankment distress and prevent an extensive failure.

21. Remedial treatment in addition to drainage methods will often be necessary when significant improvement in slope stability is required. Primary consideration should be given to constructing berms. Retaining structures for supporting slope-flattening or berm fills should be considered where right-of-way and/or suitable borrow materials are limited. A special type of retaining method (a row or rows of closely spaced piles) can be rapidly installed as a temporary or permanent (when properly designed and required to maintain traffic) support. Where embankment distress is caused largely by foundation shear failure, foundation shear trenches may be required to supplement slope-flattening or berm fills. Embankment reconstruction involving combinations of material replacement, flatter slopes and berms, and shear trenches should be considered where large settlements, shear displacements, and/or shale degradation have severely weakened embankment and/or foundation materials.

22. Specialized stabilization methods, including cement grouting and other cement, lime, and chemical treatments, may be successful under

certain conditions. Cement grouting should be considered when embankment settlements have been attributed to a high percentage of interconnected voids. Other cement, lime, or chemical treatments should be considered only on a trial basis at selected sites where risk of failure is minimal and substantial savings over more conventional remedial treatment methods can be realized. Expert guidance is required in design and application of these methods.

23. Design of remedial treatment alternatives should be based on sound geotechnical engineering principles, combined with engineering experience, judgment, and ingenuity. Design investigations should include a review of site evaluation data, past experience, and stability analyses based on in situ strengths. Stability analyses are an essential feature in the design of economical and effective remedial treatment plans. Stability analyses aid in determining the significance and interaction of design variables and provide a quantitative basis for designing remedial treatment methods consistent with engineering judgment and experience. Stability analyses should be conducted in the design of permanent or temporary support (including temporary stability of slopes excavated in construction of permanent remedial treatment). Acceptable factors of safety can vary, depending on the accuracy and confidence in design parameters and the consequence of failure. Factors of safety for permanent remedial treatment range from 1.25 to 1.5 and from 1.1 to 1.3 for temporary support. Guidance on different types of remedial methods is presented in Part IX.

PART III: FIELD EXPLORATION AND SAMPLING OF SHALES

General Objectives

24. Field exploration and sampling of shales for highway embankments should accomplish two objectives:

- a. To develop a complete picture of the geology, shale formation features, including excavation characteristics and groundwater and seepage conditions.
- b. To obtain intact, unweathered shale samples from cuts and other borrow sources for durability index and other tests.

The greater the detail of information developed for each cut and borrow source, the less will be the degree of conservatism required in assessing potential problems and the need for special design features and special construction procedures.

25. The extent of field investigations needed in the preliminary and final design phases will depend on the amount of information collected and evaluated during the corridor studies on the following:

- a. Regional geology. Extent and character of shale formations, groundwater patterns, and past problems.
- b. Local geology. Stratification variability, thickness, and inclination of strata; groundwater levels and seepage zones; joint orientation and spacing; and percentage of nondurable shale, durable shale, and harder rock in each cut and other borrow source.

26. Development of the local geology and shale formation features should include an office study of available geologic information, pertinent core logs and data from nearby projects, aerial photographs (stereographic black and white, color, and color infrared), and thermal infrared imagery. The office study should be followed by a detailed field reconnaissance along proposed routes by geotechnical engineers to obtain needed additional information such as (a) photographs and descriptions or logs of existing cut faces (roadway cuts, quarries, outcrops, and cuts along river or stream channels); (b) depth and type of field weathering in different shale beds; (c) general strike and dip of strata and frequency and spacing of joints in proposed cut areas; and (d) locations of springs and seepage zones. Based on the field reconnaissance, the shale formation features can be further defined, and the plan for borings and coring of shale in cut areas along the selected route can then be developed.

27. The extent of the field exploration program depends on the

existing information and experience in the area. For relatively uniform shale formations (such as the Kope formation in the Cincinnati area), where considerable information has been accumulated on stratigraphy and shale durability, a minimal program may be justified. However, for complex formations that change radically in aerial extent (e.g., because of folding and faulting), a thorough field investigation program may be required to define features previously listed in Table 1.

28. A key reference containing guidance on geotechnical practices is the "Synthesis of Highway Practice 33, Acquisition and Use of Geotechnical Information," Transportation Research Board, 1976.

Sources of Information

29. Sources of information for office studies, planning of geotechnical investigations, and development of geologic conditions for shale embankments include the following:

a. Geologic literature and maps. (1) U. S. Geological Survey (USGS) (regional offices in Arlington, Virginia; Rolla, Missouri; Denver, Colorado; and Menlo Park, California): "Geological and Water-Supply Reports and Maps," published for each state and updated about every five years and "Geologic Map Index," published for each state and updated about every ten years. (2) State Geologic Survey agencies: list of publications and specific inquiries (best source). (3) State highway agency: soil and geology reports and boring and rock core logs for nearby projects. (4) Corps of Engineer Districts: project design memorandum for nearby projects and specific inquiries. (5) State Geologic Society publications. (6) State University geologic reports, bulletins, theses, and dissertations.

b. Remote sensing (aerial photographs and imagery). (1) Complete listings for specific state agencies, federal agencies, and Corps of Engineers offices giving type and scale of coverage, extent of coverage, and acquisition information are given by tables in Appendix A, "Sources of Available Remote Sensor Imagery," (May 1978), to "Guidance for Application of Remote Sensing to Environmental Management," Instructional Report M-78-2 (1978). (2) A new USGS computerized index and display system provides information on planned, in-progress, and available aerial photographs of the United States. The system includes information on aerial photographs from the USGS, Soil Conservation Service, Agriculture Stabilization and Conservation Service, U. S. Forest Service, NASA, other Federal agencies, and private organizations (Eastern Mapping Center (NCIC-E), USGS, 536 National Center, Reston, Virginia, 22092; telephone number, 703-860-6336).

Aerial Photographs and Thermal Imagery for Seepage Detection

30. Aerial photographs and thermal infrared imagery provide a rapid means of defining the locations of springs, seepage zones, and drainage channels that could cause harmful saturation of shale embankments. These features should be defined during design studies and the locations of required drainage measures shown on the plans. The installation of spring drains and underdrains should not be left entirely to the project engineer on an as-required basis, since the need for such drainage measures might not be apparent during a dry construction season. He should, however, have the option of adding drains where new sources are detected during construction.

Thermal imagery

31. Thermal infrared imagery is the most useful remote method for detecting springs and seepage zones. Noble (1972)* describes a study to detect wet soils and shows examples of comparative sets of black and white photographs, daytime thermal imagery, and nighttime thermal imagery (8- to 13.5-micron wavelength band at scales of approximately 1:6,000 and 1:36,500) used in detecting soil zones of high moisture content as outlined in Table 3. He concluded that only zones of extremely high moisture content in proximity to surface water or coincident with drainageways could be detected. Stingelin (1972)** states that the detection of seepage zones (in a potential landslide study) was enhanced by infrared thermal imagery flown during late fall, at night, and with the air temperature well below the ground temperature. Under these conditions, groundwater seepage appears as white (warm) tones against a dark background, which optimizes interpretation. He also cites a reservoir study where springs and seepage zones were detected under conditions similar to the landslide study. In a more recent study, Lichy (1976)† shows comparative daytime and nighttime thermal infrared imagery (8 to 13.5 microns at scales of 1:24,000) used to detect springs, wet soil areas, sinkholes, and a shale-weathered limestone contact line. Black and white aerial photographs were also used with the thermal imagery to aid in the detection. This reference contains excellent

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* Noble, D. F., "Utilization of Remote Sensing in the Preliminary Aerial Survey-Highway Planning Stage in Virginia," Highway Research Record No. 421, Remote Sensing for Highway Engineering, Highway Research Board, Washington, D. C., 1972.

** Stingelin, R. W., "Airborne Infrared Imagery and Its Limitations in Civil Engineering Practice," Highway Research Record No. 421, Remote Sensing for Highway Engineering, Highway Research Board, Washington, D. C., 1972.

† Lichy, D. E., "Remote Sensing Demonstration Project, Verona Lake, Virginia," prepared by Baltimore District for the Office, Chief of Engineers, Washington, D. C., Dec 1976.

Table 3. Detection of High Moisture Content Soils from Remote Imagery Comparisons

Image Tone	Black and White (Panchromatic)	Daytime Thermal Infrared Imagery*	Nighttime Thermal Infrared Imagery**	Remarks
Dark tones	Dark-colored soils; vegetation	Surface water; vegetation	Low vegetation (cool features)	
	$\boxed{\text{Soils of high moisture content}}$ →	$\boxed{\text{Soils of high moisture content}}$	→	Check color and/or color IR images for presence of vegetation at dark image locations.
		Shaded ground (cool features)		
Various tones of gray	Various soil textures and other features, depending on amount of reflected light		Relatively dry soils, (depending on heat retention capacity) Trees	Dark-colored soils retain heat longer at night.
Light tones	Light-colored and relatively dry soils Roads and water surfaces, depending on reflected light	Relatively dry, dark-colored soils Roads Rock surfaces (warm features)	$\boxed{\text{Soils of high moisture content}}$ Surface water Shallow bedrock Wet depressions (sinkhole areas, fault zones, etc.) (warm features)	Shallow bedrock holds heat and can produce light tones on nighttime thermal imagery that could be mistaken for groundwater seepage without comparative study.

Coincidence of dark images on black and white photograph with dark images on daytime thermal imagery and light images on nighttime thermal imagery (at same locations) indicates presence of high moisture content material.

* Late morning.
** Late night (e.g. 8:00 p.m.) or predawn (e.g. 5:00 a.m.).

pictorial reproductions and describes the use of remote sensing for a regional and project area study including drainage, soil and geology, and fracture traces.

Required field information

32. Actual field conditions (such as soil moisture conditions, soil temperature variations, amount of water in drainage channels, location of ponds and saturated depressions, and location of shallow bedrock) and times of optimum temperature differences should be known or checked in planning the time of thermal infrared imagery coverage and also during the actual aerial survey. The influence of geological conditions such as shallow bedrock, which acts as a heat reservoir (produces light tones on nighttime infrared imagery that can be mistaken for groundwater seepage), must also be considered. Atmospheric conditions are also important in obtaining meaningful results.

Large-scale aerial photographs

33. Wet ground areas coincident with drainage channels are clearly visible on a large-scale (1:12,000) black and white photograph (taken within three days of a heavy rain) shown in the Verona Lake study (Lichy, 1976). Thus, large-scale (1:6,000 with 1 in. = 500 ft) stereographic black and white or color infrared, which enhances the contrast between vegetation and water surfaces taken shortly after significant rainfall, could also be used to detect drainage channel flow magnitudes, groundwater seepage areas, and springs if the interpreter was familiar with the topography, geology, and ground surface conditions. Springs could also be detected better on thermal infrared imagery if large-scale coverage (1:6,000) could be obtained.

Geophysical Methods

34. Geophysical methods used in field investigations for shale embankment design studies are surface seismic refraction and resistivity surveys. Surface seismic refraction surveys are useful in estimating depth of soil overlaying rock, depth to the water table above bedrock, and longitudinal (P-wave) velocities in shales and harder rock for evaluating excavation characteristics. Resistivity surveys are also useful in estimating depth to the water table and in locating near-vertical subsurface boundaries such as faults.

35. Seismic and resistivity surveys performed during initial stages of field investigations are helpful in selecting boring locations. These surveys can also be used to interpret conditions between borings. A concise summary of seismic and resistivity surveys, including relative

merits, accuracy, and limitations is given by Ash et al. (1974).*

36. Moisture-density logging of borings can also be used to detect seepage zones (from high moisture contents), depth of soil cover, and strata thicknesses (related to changes in moisture content and density). Portable nuclear moisture-density equipment is commercially available with a probe that can be lowered to depths up to 100 ft.

37. A general reference is a Corps of Engineers Manual, EM 1110-1-1802, "Geophysical Explorations," dated 28 February 1979. This manual covers all types of geophysical methods and was rewritten in 1977-78.

Seismic refraction surveys

38. Acquisition of high quality seismic refraction data requires experienced personnel, adequate equipment, and a properly designed field program. Major factors that influence results are layout of a spread (such as the 12- or 24-geophone spread with five impact or shot points), topography of the site, and geophone placement.

39. Seismic refraction surveys using single-channel hammer apparatus, while portable and simple to operate, have several disadvantages. A strong, uniform hammer impact is critical to the success of the survey. The seismic energy is not always uniform at different source and receiver locations. The apparatus cannot be used on soft ground, and the depth of investigation is limited to about 90 ft. Since signals must be picked without proper consideration of travel times for different hammer-to-geophone distances, the signal is often covered by noise, and second events cannot be determined.

40. The advantages in using a multichannel unit with an explosive energy source are (a) an energy source common to all geophones, (b) unlimited depth of investigation, (c) large area of coverage in a relatively short time, and (d) data can be permanently recorded, allowing noise energy to be distinguished from refraction energy and sometimes second events to be recognized and used in interpreting problems such as the hidden layer. Such factors as geophone spread, geophone spacing, size of charge, depth and tamping of shot, geophone placement, amplifier gains, and filter settings are important in obtaining valid data.

41. A key reference by Greenhalgh and Whiteley (1977)** gives a

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* Ash, J. L. et al., "Improved Subsurface Investigations for Highway Tunnel Design and Construction, Vol. 1, Subsurface Investigation System Planning," Report No. FHWA-RD-74-29, Federal Highway Administration, Washington, D. C., May 1974.

** Greenhalgh, S. A. and Whiteley, R. J., "Effective Application of Seismic Refraction Methods to Highway Projects," Australian Road Research, Vol. 7, No. 1, Mar 1977.

detailed review of equipment criteria, field procedures, and interpretation techniques. A field example from this reference is shown in Figure 5.

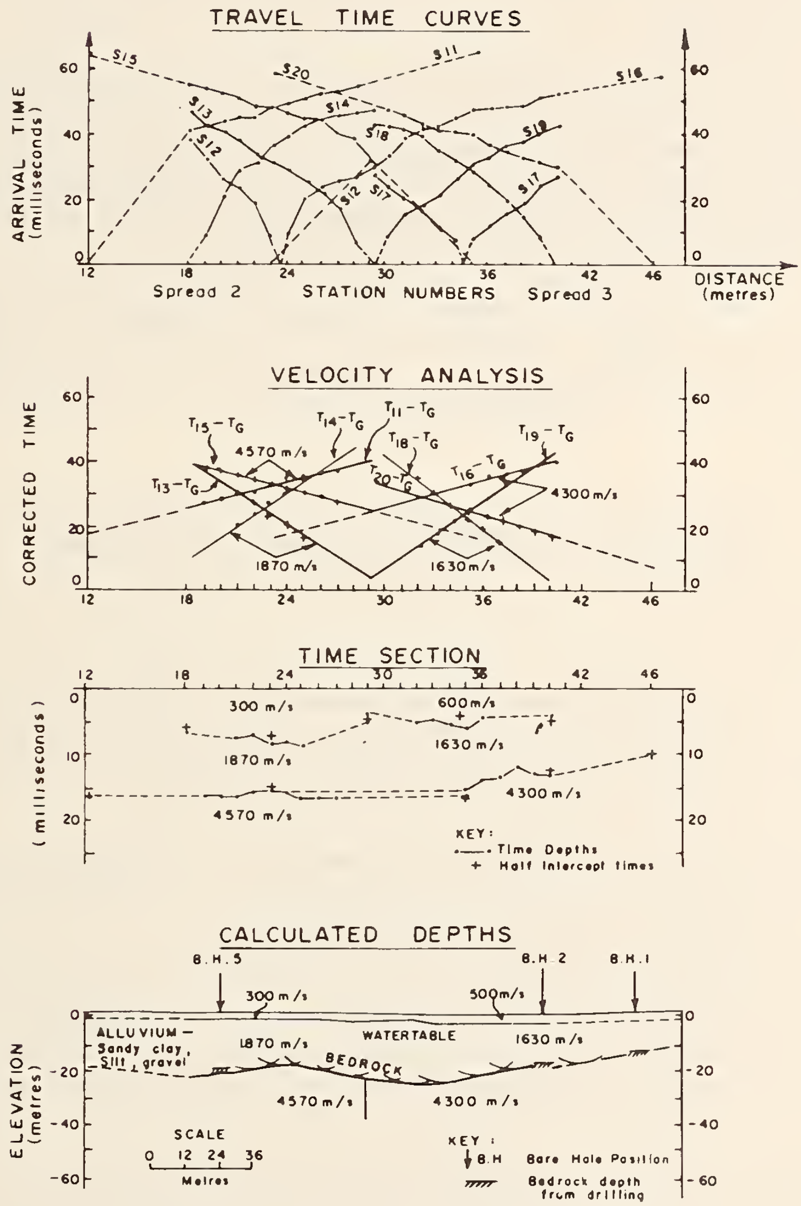

Figure 5. Cross section along seismic traverse, showing bedrock profile and drill locations together with time distance curves, corrected travel times, measured velocities, and time depths (Greenhalgh and Whiteley, 1977).

42. Because of the usual complex nature of shale formation, such as alternate layers of harder (higher velocity) and softer strata (lower velocity), seismic surveys should not be depended on to determine depths to various layers. Since coring of shales is required to obtain shale samples, core logging should be the primary means of defining stratification.

Resistivity surveys

43. Resistivity surveys are often more sensitive to shallow variations in weak superficial deposits and to groundwater conditions than seismic surveys, especially when low velocity contrasts and velocity inversion with depth exist. Resistivity surveys should also be considered when practical limits of geophone spacings and travel time measurements cannot be obtained in seismic surveys.

44. In resistivity surveys, the Wenner configuration appears to be the best arrangement. Higginbottom (1976)* describes resistivity surveys and their interpretation. Stephens (1973)** describes the performance of different resistivity equipment and curve-fitting techniques.

Groundwater and Surface Drainage Observations

Groundwater

45. While springs and seepage zones that could affect shale embankments can be located by remote sensing and field reconnaissance, the amount of water to be drained requires ground observation. Otherwise, the selected drainage system may be inadequate.

46. Springs and groundwater seepage exits that would be covered by the embankment or that are located upslope on sidehill locations should be monitored during the field boring program and field reviews (especially during winter months where ice buildup is a prominent sign of seepage). Selected borings in the embankment foundation near the cut-fill transition and in the uphill toe area of sidehill embankments should be cased and water level readings taken after periods of heavy rainfall to determine maximum levels. Simple tests for flow quantities

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* Higginbottom, I. E., "Part 2: Electrical Resistivity, Magnetic and Gravity Methods, Engineering Geology in Practice in Britain: 7," Ground Engineering, Mar 1976.

** Stephens, E., "Electrical Resistivity Techniques," Final Report, Transportation Laboratory Research Report CA-DOT-TL-2102-1-73-35, State of California, Department of Transportation, Division of Highways, Sacramento, CA, Dec 1973.

can be estimated by bailing out the cased hole to reduce the water level several feet and determining the time for refilling to the original level. Springs and surface seeps should be ditched and an outlet formed where flow quantities can be measured. For major embankments (generally higher than 50 ft), piezometers installed and sealed in permeable (sandstone and limestone) strata can be used to obtain seepage locations and variations in piezometric levels before and after periods of heavy rainfall.

Surface drainage

47. Hills and ridges sloping into the embankment toe line above a main drainage channel can produce significant drainage. Paved ditches along the embankment toe line may be inadequate to carry the runoff unless the locations and need for increased drainage capacity are noted during field investigations. Otherwise, the paved ditch lines will be eroded and water will infiltrate into the embankment.

Development of Geology

48. Development of the geology in shale formations along the selected route can be accomplished during the preliminary stage from geologic quadrangle maps, supplemented by a detailed field reconnaissance, and core borings in areas where there are no outcrops or existing cuts. An example for a short segment of a center-line profile along I-74, with the geology added from "Bedrock Geology of the Addyston Quadrangle and Part of the Burlington Quadrangel, Hamilton County, Ohio," (Ford, 1972)* is shown in Figure 6. The information for the Fairview and Kope formations gives a description of the shales, bedding thicknesses, and ratio of shale and limestone layers. This information is useful in assessing the feasibiltiy of selective excavation and placement during construction, planning the location of core holes, and determining the quantity of shale samples needed for testing.

49. In this simplified example of horizontal bedding with shales known to have a low slake-durability index and requiring compaction as soil, representative shale samples would be needed for compaction tests (see paragraph 85). Thus the quantity of shale to be sampled from each cut could be defined prior to the field boring and coring program. In other more complex formations, a core boring may be required in each cut during the preliminary phase to refine the geology and obtain shale samples for durability tests unless unweathered shale samples can be obtained from existing cuts and outcrops.

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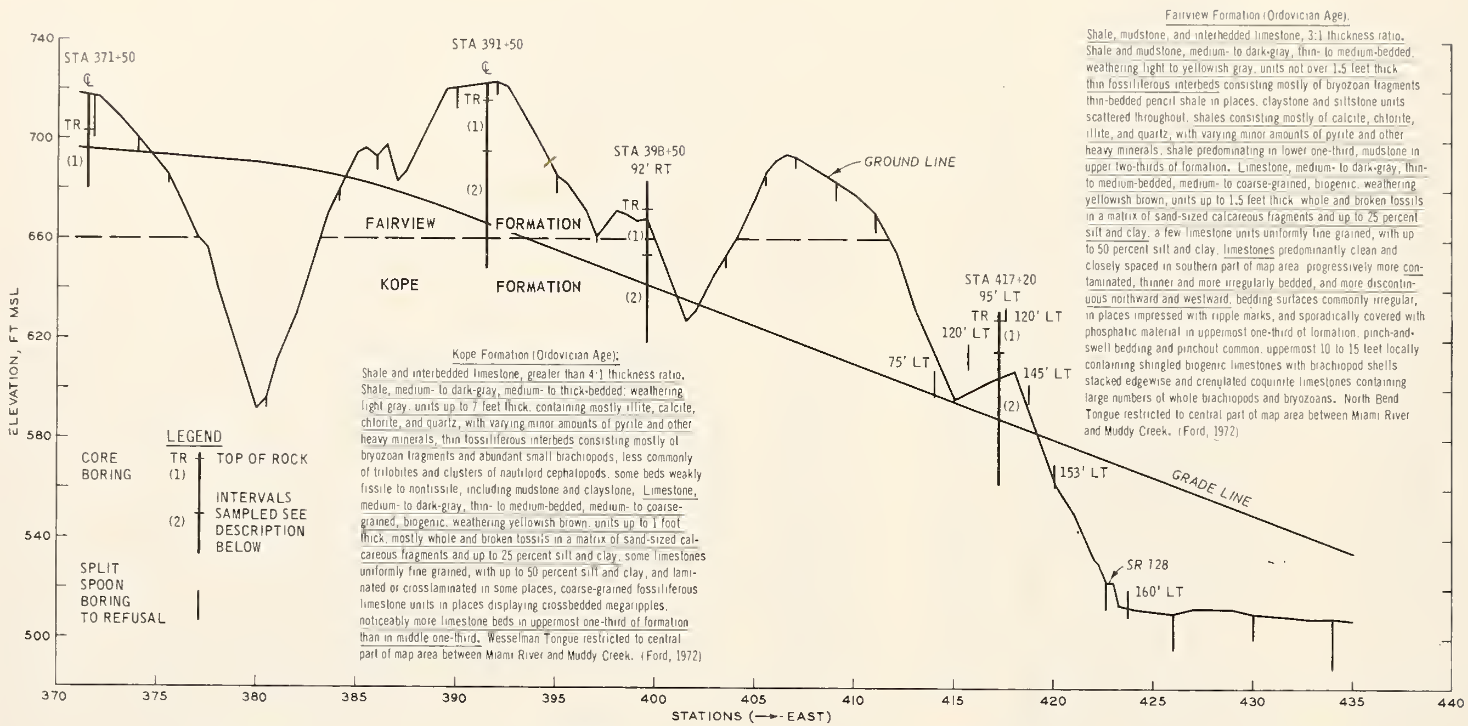
* Ford, J. P., "Bedrock Geology of the Addyston Quadrangle and Part of the Burlington Quadrangle, Hamilton County, Ohio," Ohio Department of Natural Resources, Division of Geological Survey, Fountain Square, Columbus, OH, 1972.



Sta 371+50

(1) Shale, brown (predominant) and grey, argillaceous, somewhat calcareous, generally medium firm but varies from soft to firm, interbedded with limestone (grey, crystalline, hard, fossiliferous, firm, broken el 686 to 684, comprises 19% of interval in layers 1/2 to 7-in. thick, averaging 2 to 3-in. thick).
Core loss: 11%

Sta 391+50

(1) Shale, brown with some grey layers, argillaceous, somewhat calcareous, soft, interbedded with limestone (grey, crystalline, fossiliferous, hard and firm. comprises 43% of interval in 1 to 6-in. layers).
Core loss: 29%.

(2) Shale, grey, argillaceous, somewhat calcareous, firm interbedded with limestone (grey, crystalline, fossiliferous, hard, firm, comprises 16% of interval in layers 1/2 to 5-in. thick).
Core loss: 7%

Sta 398+50

(1) Shale, brown, extremely weathered and crumbly, soft calcareous, with thick clay seams throughout, interbedded with limestone (grey, crystalline, fossiliferous, hard, in 1 to 3-in. layers).
Core loss: 60%

(2) Shale, grey argillaceous, somewhat calcareous, firm, interbedded with limestone (grey, crystalline, fossiliferous, hard, firm, comprises 18% of interval in layers 1 to 8-in. thick). Between el 649 and 640 (approximately) and el 629 and 624 thin bedded layers are broken (cavernous?). Pockets of brown silty sand containing a small amount of limestone gravel occur at these intervals.
Core loss: 30%.

Sta 417+20

(1) Shale, brown, extremely weathered, crumbly, soft, calcareous, with thick clay seams throughout, interbedded with limestone (grey, crystalline, fossiliferous, hard, in layers 1 to 3-in.).
Core loss: 25%.

(2) Shale, grey, argillaceous, somewhat calcareous, fossiliferous, hard to generally firm, interbedded with limestone (grey, crystalline, fossiliferous, hard to firm, comprises 9% of interval in layers 1 to 5-in. thick).
Core loss: 25%.

Figure 6. Example of geologic profile using existing quadrangle geology and boring logs (discussed subsequently). (Boring log description courtesy of Ohio Department of Transportation)

50. A geophysical survey may be helpful in determining the depth of soil cover and general groundwater depth between borings. Seismic velocities are useful for estimating excavation characteristics (see Part V). However, during the field boring and coring program as much information as possible should be obtained on joint spacing and orientation, fracturing, and hardness of shales and other rock types to aid in assessing excavation difficulty.

51. The geology can be refined from field borings and coring of shales. The soil and geology conditions should be shown on profiles either by descriptive boring logs (bottom of Figure 6) or graphically as in Figure 7. Information on the percentage of harder rock layers, thickness of these layers, and elevation intervals of thicker shale beds should be included to aid in the final decision on the excavation and placement methods and compaction procedures that will be required during construction.

Borings and Sampling Requirements

52. The location and number of borings to obtain cores of shales and harder rock should be planned on a cut-by-cut basis, depending on the complexity of the formation, available information, and previous experience. Where bedding is known to be horizontal and the depth of soil cover has been determined from shallow auger borings and geophysical surveys, core borings to grade at 200-ft intervals in each cut may be adequate to check strata thickness, percentage of shale and harder rock, and depth to the water table. Additional coring may be required in softer shales, which are to be compacted as soil, to obtain a sufficient quantity of shale for laboratory compaction tests.

Examples of boring locations

53. Where stratigraphic conditions are not well defined, the minimum number of core borings in each cut may range from one to two for each 200 ft of center-line distance. Suggested spacing and depth in a cut with relatively horizontal strata is shown in Figure 8. In this case, shales are nondurable and shale samples (cores) are needed mainly for laboratory compaction testing (e.g., varying from 58 to 260 lb of the four shale layers). The inclination of layers, as determined during field reconnaissance or during coring operations may require coring to a significant depth below grade. The depth should be adequate to define seepage direction into fill areas where underdrains would be required. Where shale type and stratigraphy are fairly uniform along the project, the number of core borings in other cuts could be reduced to one or two for shale sampling and checking of stratigraphy and percentage of harder rock.

54. A complex example of steeply inclined bedding is shown in Figure 9. Eight core borings are needed to sample the majority of strata. Without some prior information on dip and strike of strata from

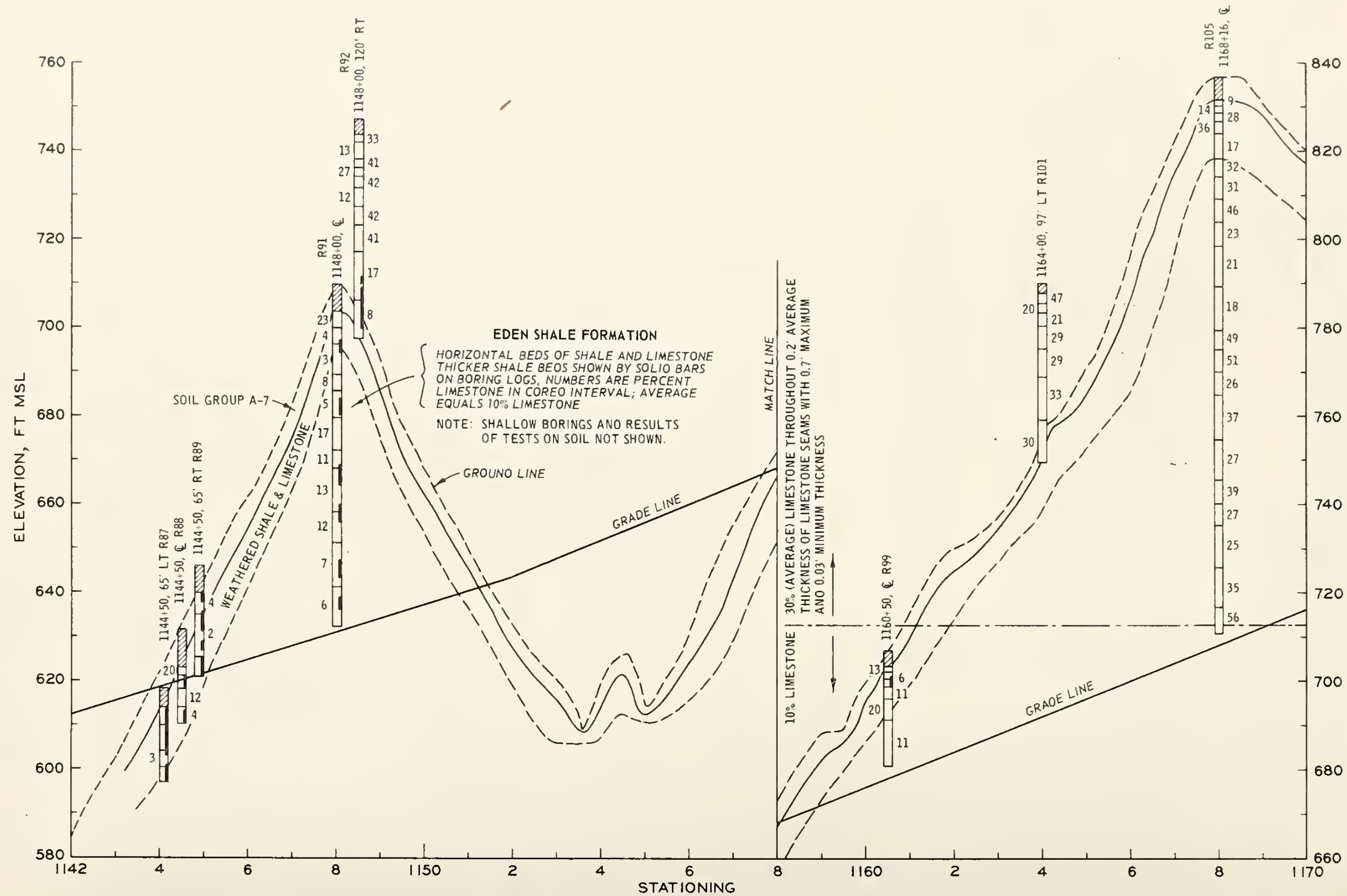


Figure 7. Example of geologic profile from boring logs (courtesy of Kentucky Bureau of Highways)

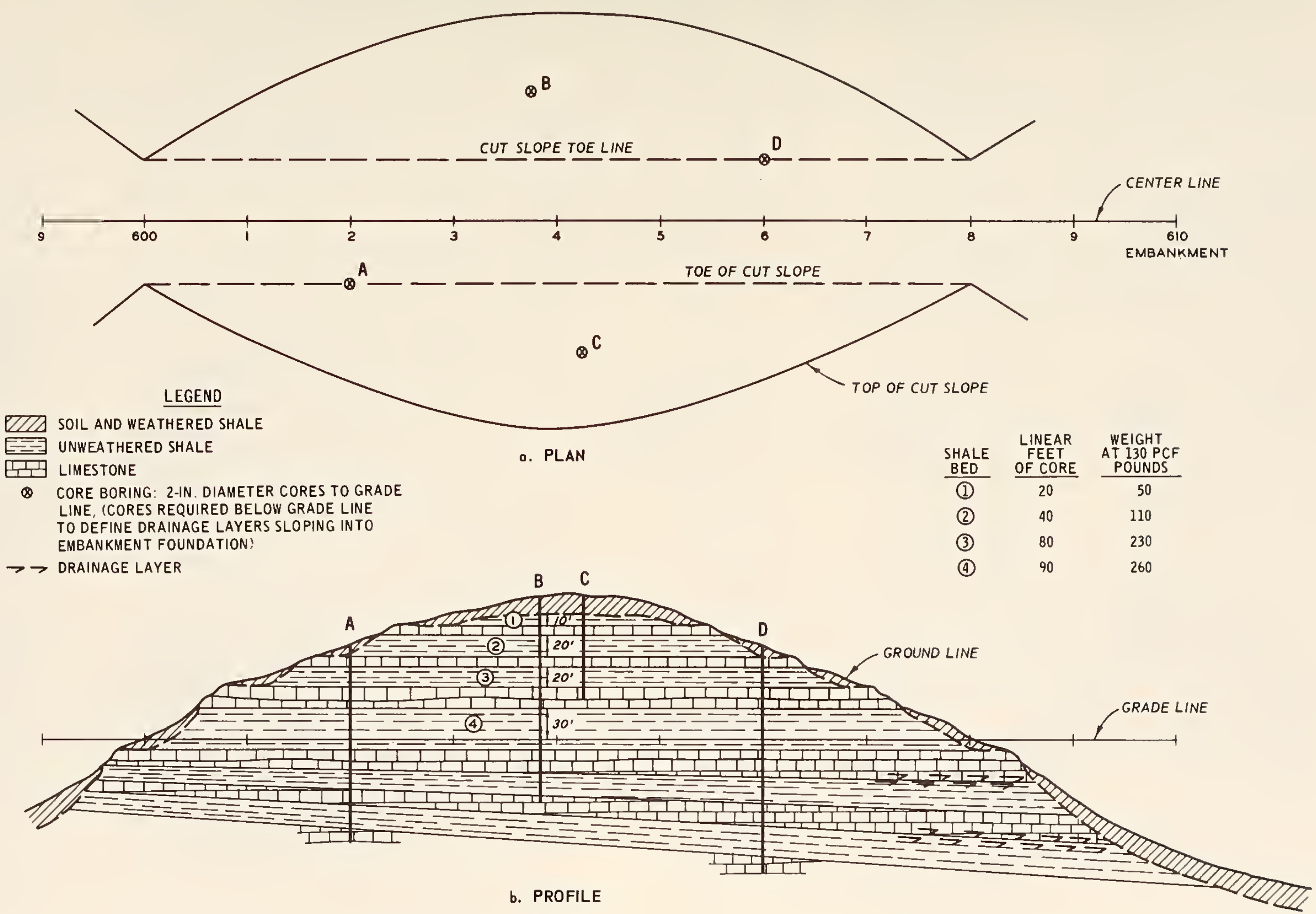


Figure 8. Example of core boring locations for sampling horizontal shale strata

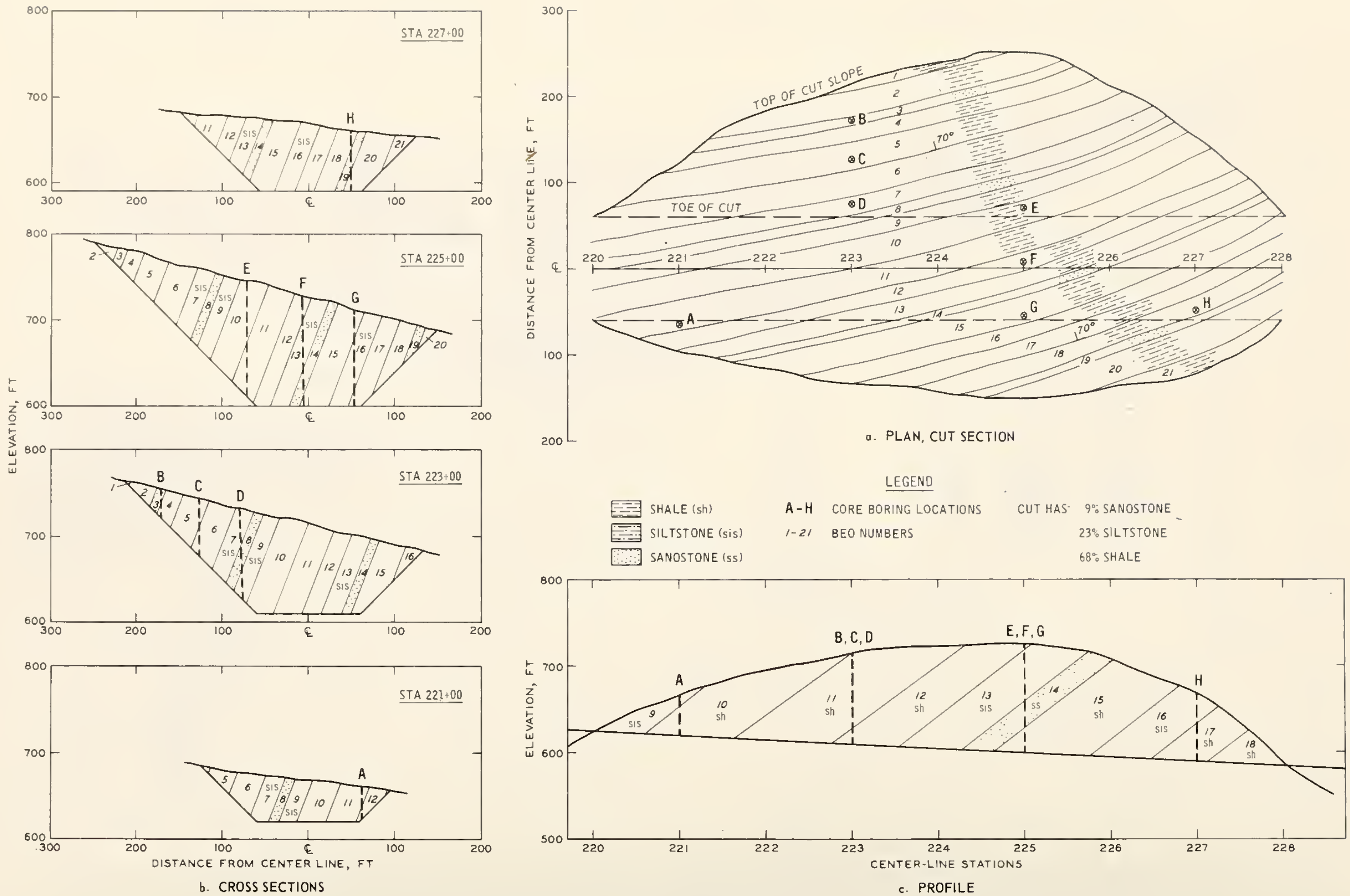


Figure 9. Example of core boring locations for sampling steeply dipping shale and siltstone strata

the field reconnaissance, it would be difficult to develop an economical plan of boring locations and depths. As an alternative, inclined core borings could be used to reduce the total number of borings.

55. The experience and judgment of the geotechnical engineer should influence the design studies. In the example of steeply inclined strata, unweathered shale samples obtained by hand-excavation during the field reconnaissance may have indicated low slake-durability indexes. Thus a decision to break down and place all material in the cut as soil would reduce the sampling requirements to that needed to check representative strata for low slake durability and to obtain samples of representative shales for compaction testing.

Drilling procedure

56. Core borings in shale should be made to obtain, as a minimum, 2-in.-diameter cores. Coring with compressed air is preferable to water or drilling mud to prevent excessive wetting of softer shales. Coring with air is discussed by Thompson (1971).* Casing of the hole to the bottom of the overburden soils may be required when using compressed air.

Logging of cores

57. Complete information on in situ shale and harder rock should be included on core boring logs. Examples of descriptive logs are shown in Figures 6 and 10. A suggested list of items and descriptive terms is shown in Table 4.

Preserving shale cores

58. Shale cores are usually the primary source of samples for laboratory tests. Compaction tests (and point load tests, if used) require shale material that has not dried out. Many shales, if allowed to dry, will slake or break apart when rewetted. Even a small loss of moisture affects some shales (especially laminated, clayey, and silty shales). Dried out shales, rewetted for laboratory compaction tests, will degrade or soften. The compaction test results would not represent field conditions unless a similar amount of drying and rewetting occurred during construction. Also, the natural water content is an important aid in identifying nondurable shales, soft shales, and possible seepage zones. Consequently, all shale cores needed for laboratory tests should be preserved to prevent drying as soon as they are removed from the sampler. Early identification of nondurable shales is necessary so that sample requirements (Table 5) for these shales can include

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* Thompson, B. L., "The Use of Air as a Drilling Medium for Subsurface Investigations," Proceedings, 22nd Annual Highway Geology Symposium, Oklahoma Department of Highways, Oklahoma City, OK, Apr 1971.

EXAMPLE EXAMPLE

ROCK CORING RECORD

County PIKE

Project No. APD 61 (15);SP 67-35-21

Project Name HAZARD-PIKEVILLE ROAD

Core Location 160' Rt., Sta. 221 + 00

Surf. Elev. 1000

Date Started September 15, 1968

Rock Core Dia. BX (1 5/8") and NX (2 1/8")

Date Completed September 16. 1968

Driller I. R. FAST

Page 1 of 1

Elevation	Depth	Description of Material	Coring Run	Core Recovery	Core Loss	% Recovery	Remarks
1000.0' 995.0'	0.0	Silty brown top soil to 1'.0"; chert fragments from 1'.0" to 5'.0".				0%	Roller bit used to 5.0'.
990.0'	10.0'	Broken, gray limestone very hard, crystalline	5.0'	4.8'	0.2'	96.0	Weathered vertical solution channel
980.0'	15.0' 20.0'	Very hard, gray L. S. Coarse and crystalline frequent chert layers.	10.0'	10.0'	0.0'	100%	1" chert layer average approx. 1'.0' apart.
970.0'	25.0' 30.0'	Soft, green shale with thin (1" to 2") shaly L. S. layers (30 % of interval)	10.0'	6.0'	4.0'	60.0%	Shale dries and crumbles badly.
960.0'	35.0' 40.0'	Very hard, brown siltstone with thin shale stringers, well cemented with calcite.	10.0'	9.5'	.5'	95.0%	Void area from 35.0' to 35.5'. Lost drill water.
950.0'	45.0' 50.0'	Very hard, coarse grained S. S. with large quartz pebbles. Cross-bedded from 48.0 to 50.0.	10.0'	9.8'	0.2'	98.0%	Used two core bits on this run.
940.0'	55.0' 60.0'	Hard, black shale with pyrite crystals. Bottom of Core.	10.0'	10.0'	0.0'	100.0%	Cored extremely hard.
935.0'	65.0' 70.0'	Light gray, fine grained L. S. with thin shale partings.					Open face log from elevation 940 to 935.0'

Figure 10. Example of boring record (courtesy Kentucky Bureau of Highways)

Table 4. Suggested Terms for Describing Materials Cored from Shale Formations

Item	Descriptive Terms
Type of Material	Shale (thinly laminated or fissile), clay shale, claystone, siltstone, sandstone, limestone, etc.
Color	General color: gray, brown, black, reddish-brown, green, light gray, dark brown, etc.
Lithologic characteristics	Clayey, silty, sandy, shaly, carbonaceous, calcareous (limy), bentonitic, clay seams, silt partings, bentonite stringers, fossiliferous, crystalline (for limestones), concretions, nodules, etc.
Degree of weathering	Slightly weathered (discontinuities such as joint surfaces slightly discolored)
	Weathered (joint surfaces discolored, alteration penetrating inward)
	Highly weathered (mass discolored, alteration penetrates deeply but "core stones" present
	Completely weathered (changed to soil, but original fabric mainly preserved, some "core stones" present
Hardness	Soft (can be pealed with a knife or scratched with fingernails)
	Crumbly (crumbles under finger pressure)
	Moderately hard (can be easily scratched with a knife)
	Very hard (cannot be scratched with a knife)
Cementation	Cemented (calcareous cementing material between particles forming visible grains that do not reduce to powder when scraped with a knife)
Texture (sandstones, mainly)	Dense (smooth appearing)
	Fine-grained (grains just visible to eye (fine sand size))
	Medium-grained (0.4 to 2mm)
	Coarse-grained (2 to 5 mm coarse sand size)

Item	Descriptive Terms
Bedding Characteristics	Massive (greater than 3-ft thickness)
	Thick (3 ft to 1 ft)
	Medium (1 ft to 4 in.)
	Thin (less than 4 in.)
	Crossbedded (sandstones)
Interbedding of harder rocks	Type of rock, layer thickness, spacing, description, and percent of cored interval (e.g. 2- to 8-in. layers averaging 4 in. constitutes 16% of interval); otherwise note each layer, give thickness and describe
Structure	Bedding (flat or dip in degrees)
	Fractures (give spacing or note as: slightly fractured (3 to 6 ft); moderately fractured (1 to 3 ft); highly fractured (4 in. to 1 ft); intensely fractured (less than 4 in.))
	Joints (discolored surfaces). Note interval and dip and any infilling with clay or minerals
	Faults (suspected from core loss, ease of drilling, and returned cuttings)
	Voids (interval drill rods drop, sudden loss of drilling fluid or air pressure)
Slaking Properties	Severe (breaks apart at laminations or into small pieces on exposure to air or disintegrates after air drying and immersion in water)
	Moderate (breaks apart at laminations, softens, or breaks apart into small pieces after air drying and soaking in water)

Table 5. Type and Quantity of Shale Sample Required for Different Laboratory Tests

Laboratory Test	Type of Representative Shale Sample	Initial Sample Condition	Quantity of Sample/Test	Approximate Amount of Core per Test
Natural Water Content	Chunks or core pieces	At in situ water content (not dried out)	Minimum of 50 g, or several 1 to 2-in. size pieces	1 to 2 in. of 2-in.-diameter core
Jar-Slake	Unweathered, undisturbed chunks or core pieces	Can be dried out (unless used initially for natural water content test)	Minimum of 80 g, or 2- to 3-in. size pieces	2 to 3 in. of 2-in.-diameter core
Slake Durability	Unweathered, undisturbed chunks or core pieces	Can be dried out (unless initially used for natural water content)	400 to 600 g, minimum of 10 rounded chunks each 40 to 60 g (1 to 1-1/2-in. size pieces)	About 2 ft of 2-in.-diameter core
Slake	Unweathered, undisturbed chunk or core pieces	Can be dried out (unless used initially for natural water content test)	150-g chunk (3- to 4-in. size pieces)	3 to 4 in. of 2-in.-diameter core
Rate of Slaking	Unweathered, undisturbed chunk or core pieces	Can be dried out (unless used initially for water content test)	Several small (less than 1-in. size) pieces	1 to 2 in. of 2-in.-diameter core
Compaction (nondurable shales)	Unweathered, undisturbed chunks or core pieces	At in situ water content (not dried out)	15 kg (33 lb) for 3 point tests (about 45 lb of unprocessed shale)	10 ft of 2-in.-diameter core

the extra material needed for compaction tests.

59. One way of preserving shale cores is to roll up the core in a heavy (0.3- to 0.5-mil) plastic bag and seal the end of the bag. Plastic sheeting wrapped around the core and sealed with tape is an alternate method. The cores remain visible for logging.

Defining Special Problem Areas

60. Special problem areas for shale embankments include (Vol. 2):

a. Sidehill locations on deep deposits of colluvium or weathered shale having low shear strengths.

b. Sidehill locations on shale and limestone or sandstone layers dipping downslope where seepage water would be blocked by the embankment.

c. Sidehill locations along landslide-prone slopes.

d. Springs and surface seepage zones along sidehill locations or in cut-fill transition areas.

e. Steeply inclined beds of soft shale or hard nondurable shale interbedded with harder rock where excavation by blasting would be required to break down all materials for compaction as soilfill.

61. These problem areas should be well defined before the final alignment is established. Where the final alignment or grade cannot be shifted to avoid or minimize the problem, extensive stabilization measures such as deep drainage trenches and/or horizontal drains or retaining structures may be required on hillside locations to avoid a costly failure after construction. Complex geologic conditions in cuts may require extra blasting to reduce all materials to an acceptable size for compaction as soil in thin lifts. In a cut-fill transition area, spring drains, subdrains, and benching may be required to drain excess seepage from pervious strata that are dipping out of the foundation surface.

62. Along sidehill problem locations deeper than normal borings, rock coring and sampling may be required over a wider area to define depths of weak materials, stratification sequence, bedding inclination, and groundwater seepage conditions that will affect the stability of the foundation area. In geologically complex cuts, more extensive explorations and sampling may be needed to define stratification sequence, bedding orientation and inclination, spacing of joints, fractures and bedding planes, groundwater depths, and shale durability.

Graphical Display of Geotechnical Information

63. Soil and geologic profiles and sections should be developed to graphically indicate the in situ conditions and summarize foundation properties (classification, water contents, and shear strengths) and excavation material properties (degree of excavation difficulty, durability index of shales, and compaction properties of soillike materials). This type of geotechnical information format can show the problems and serve as a basis for developing required design features and special construction criteria in a much better way than a series of separate boring logs and descriptions.

64. As a minimum, existing soil and geologic conditions should be shown graphically in plan and profile and preferably include sections through cuts and on sidehill locations, as shown in Figures 6 and 7 of the NCHRP synthesis 33.* An example for a large cut is shown in Figure 11. Development of geotechnical information has several benefits as outlined by Royster (1973):**

- *a*. Provides better design information for fitting the design to all conditions in the field, rather than relying on the construction forces to attempt to fit the materials and conditions in the field to the design.
- *b*. Aids the project engineer in recognizing various materials and conditions as they are uncovered.
- *c*. Gives the contractor a better idea, during the bidding stage, of the excavation characteristics of materials to be encountered and the required construction procedures.

Without inclusion of geotechnical conditions in plan, profiles, and sections as part of the construction plans, the advantage to potential bidders is lost.

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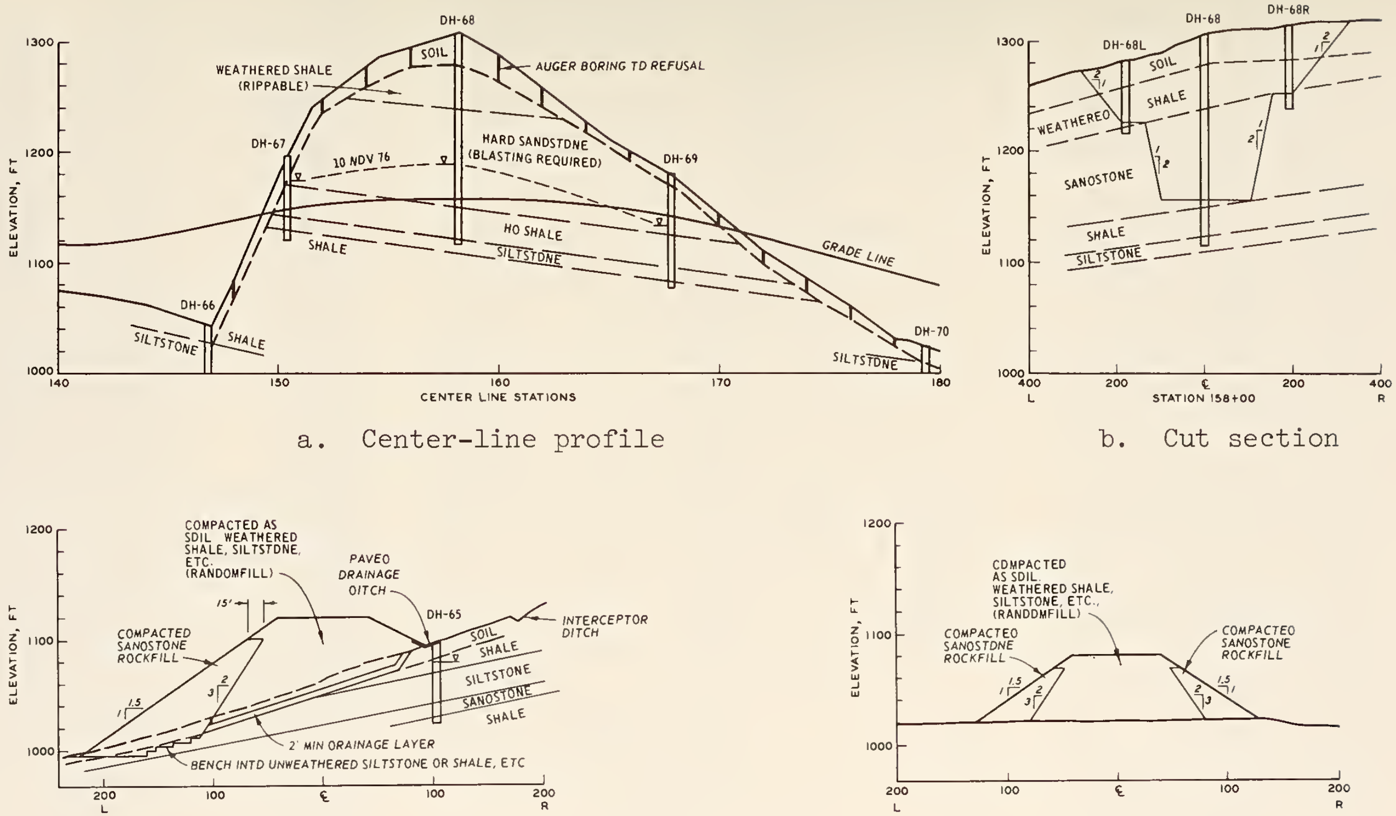
* Transportation Research Board, "Acquisition and Use of Geotechnical Information," Synthesis of Highway Practice 33, National Cooperative Highway Research Program, Washington, D. C., 1976.

** Royster, D. L., "The Role of the Division Soils and Geological Engineer in the Construction and Maintenance of Tennessee's Highways," Proceedings of the 54th Annual Tennessee Highway Conference, Bulletin No. 39, Engineering Experiment Station, The University of Tennessee, Knoxville, TN, Jan 1973.



a. Center-line profile

b. Cut section

c. Embankment section, sidehill and cut-fill transition (benching, drainage layer, and sandstone shells)

d. Embankment section, through fill

NOTE: COMPLETED DRAWING SHOULD INCLUDE

a. ENGINEERING PROPERTIES OF SOIL AND WEATHERED SHALE INCLUDING COMPACTION DATA (AASHTO-T99).

b. GRADATION CRITERIA FOR EXCAVATED MATERIALS.

c. COMPACTION CRITERIA FOR RANDOMFILL AND ROCKFILL (LIFT THICKNESS, MAXIMUM ROCK SIZE AND PERCENT COMPACTION OR PROCEDURAL CRITERIA INCLUDING TYPE COMPACTION EQUIPMENT MINIMUM WEIGHT, NUMBER OF COVERAGES OR REFERENCE TO SPECIAL PROVISIONS).

Figure 11. Geotechnical center-line profile and sections

PART IV: INDEX TESTS AND DURABILITY CLASSIFICATION CRITERIA FOR SHALES

65. The most important aspect in the design and construction of shale embankments is distinguishing durable shales that can be used in rockfill from nondurable shales that must be placed and compacted as soil. Shales that are relatively soft and nondurable are easily identified by simple slaking tests. Other shales that are mechanically hard and durable can also be recognized by their resistance to weathering in the field; dense, fine-grained appearance; ringing and resistance to breaking under hammer blows; and unchanging nature when subjected to slake-durability testing. However, other shales that are mechanically hard, yet nondurable, are more difficult to define and present major problems during construction since they are hard to break down and compact.

Index Tests

66. Numerous index tests have been proposed to assess shale durability (Chapman, 1975).* Based on Chapman's comparative studies, the review in Vol. I, Phase I, and the study reported in Vol. 3, Phase III, in which embankment performance data is related to the slake-durability index, the following simpler index tests are recommended:

- a. Jar-slake test.
- b. Slake-durability test.

An alternate test is the slake test (Chapman, 1975), and a supplementary test is the rate of slaking test (Morgenstern and Eigenbrod, 1974).**

Jar-slake test

67. The jar-slake test is qualitative with six descriptive degrees of slaking determined from visual observation of ovendried samples soaked in tap water for 24 hours. The six values of the jar-slake index, I_J , are listed below:

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* Chapman, D. R., "Shale Classification Tests and Systems: A Comparative Study," Joint Highway Research Project, JHRP-75-11, Purdue University, Jun 1975.

** Morgenstern, N. R. and Eigenbrod, K. D., "Classification of Argillaceous Soils and Rock," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 100, GT10, Oct 1974.

I_J	Descriptive Behavior
1	Degrades into a pile of flakes or mud
2	Breaks rapidly and/or forms many chips
3	Breaks rapidly and/or forms few chips
4	Breaks slowly and/or forms several fractures
5	Breaks slowly and/or develops few fractures
6	No change

Reaction to the jar-slake test usually occurs within the first 10 to 30 minutes, and a standard of 24 hours is recommended as a convenient maximum time for initial testing of a large number of samples. As experience is gained with shales in a particular formation, the maximum time can be reduced to 2 hours or less.

Slake-durability tests

68. The slake-durability test is performed on 10 pieces (with corners rounded to minimize mechanical breakage) of ovendried shale (40 to 60 grams each) submerged and rotated in a wire drum cage (No. 10 screen) at 20 rpm for 10 minutes. The procedure is repeated on the material retained in the drum after ovendrying. The two-cycle slake-durability index, I_D, is the percent of ovendried material retained after the test.

$$I_D = \frac{\text{Dry weight after two cycles}}{\text{Dry weight before testing}} \cdot 100$$

Details of the test apparatus and procedures, given in Vol. 1 and 3, are those for the standard test adopted by the International Society for Rock Mechanics (1972).*

69. The test apparatus can be obtained commercially or can be made

* International Society for Rock Mechanics, Commission on Standardization of Laboratory and Field Tests, "Suggested Methods for Determining Water Content, Density, Absorption and Related Properties and Swelling, and Slake-Durability Index Properties," Committee on Laboratory Tests, Document No. 2, Final Draft, November 1972 (U. S. National Committee for Rock Mechanics, National Research Council, 2101 Constitution Avenue, NW, Washington, D. C. 20418).

(Chapman, 1975). The commercial apparatus (Engineering Laboratory Equipment Limited, Hemel Hemstead, Hartfordshire, England; no current U. S. representative) costs approximately $1500. Locally made equipment is being used in Indiana and Kentucky.

Rate of slaking test

70. The rate of slaking test, as discussed by Chapman (1975), is a useful supplementary test, performed on an ovendried piece (less than 1-in. size) soaked two hours and drained in a funnel containing filter paper. The initial in situ water content and soaked water content (difference is Δw), plastic limit (PL), and plasticity index (PI) are required to calculate the change in liquidity index (ΔI_L), which is related to the rate of slaking (S_R) as shown in the following tabulation.

$\Delta I_L = \frac{\Delta w}{PI}$	S_R
< 0.75	slow
0.75 to 1.25	fast
>1.25	very fast

Rate of slaking tests can be performed in connection with jar-slake tests if the in situ natural water content has been measured.

71. The rate of slaking test for harder shales requires considerable effort to break down the shale for the Atterberg limit tests. Cyclic drying, soaking, and crushing in a soil pulverizer may be required to disaggregate hard shales into silt and clay particles. Softer shales that tend to slake or soften when ovendried and soaked can be disaggregated using a food blender. Sensitivity of the PI to the degree of pulverization controls the validity of this test.

Slake test

72. The slake test (Chapman, 1975) consists of five cycles of ovendrying a 150-gram sample for 8 hours and soaking it for 16 hours. After soaking, the sample is washed over a No. 10 sieve and the retained material ovendried before the next soaking. The test is performed on six samples. The slake index is the percent of total meterial lost during the test, averaged for the six samples:

$$I_S = \frac{DW_i - DW_r}{DW_i} \cdot 100, \text{ average for six samples}$$

where

DW_i = initial dry weight of sample

DW_r = dry weight of material retained on No. 10 sieve after five cycles

Classification Criteria for Shales

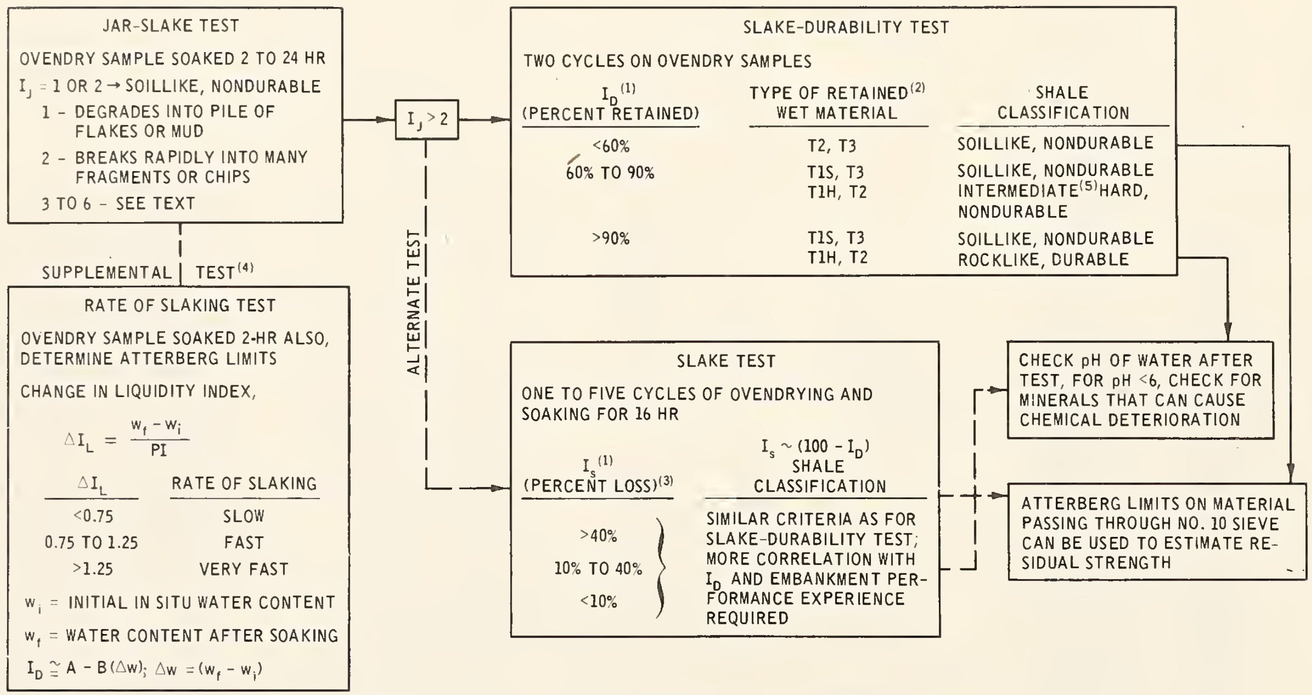
73. Shales for highway embankments should be classified as soillike, or nondurable, and rocklike, or durable. Figure 12 shows a recommended guide for index tests and classification criteria. The criteria may need to be modified based on local experience and shale embankment service performance.

74. The jar-slake test is recommended as the basic screening test. Shales with a jar-slake index, I_J, of one or two obviously should be considered soillike without further testing. The slake-durability test is considered as the main index test for shales.

75. The jar-slake test (paragraph 67) can be extended to provide more useful information by describing the behavior and hardness of soaked pieces when attempts are made to indent with the fingernail and break apart, crumble, or snap in two with the thumbs and fingers. Many shales that do not slake can be broken apart or crumbled. This softening indicates that the shale could be crushed under rock contacts in an embankment if soaking occurred.

76. The rate of slaking test (paragraph 70) can be a useful supplement to detect hard shales with a fast slaking rate, S_R. This was true for a hard shale studied by Chapman (1975). The hard shale (Borden group) had a slake-durability index of 98 but showed a ΔI_L value of 1.03, indicating a fast rate of slaking. The change in water content, Δw, measured as part of the rate of slaking test is also useful. For hard, dense, rocklike shales, Δw is usually less than one percent, but for the hard shale just cited, Δw was 4.8 percent. Chapman (1975) found that Δw was also related to I_D (Figure 12). Consequently, the rate of slaking tests, even if simplified to measure only the water content increase after soaking (compared to the in situ natural water content), could be very useful in detecting shales that could weaken with time in an embankment where soaking by water infiltration could occur.

77. As indicated in Figure 12, shales with $I_J > 2$ should be subjected to the slake-durability test. The range of I_D values and the type of breakdown and hardness of the retained material are important in determining whether the shale should be considered soillike. The I_D criteria values shown in Figure 12 are somewhat conservative compared to the embankment performance and lift-thickness information in



NOTE: (1)DIFFERENT LIMITING VALUES MAY BE JUSTIFIED ON BASIS OF LOCAL EMBANKMENT PERFORMANCE EXPERIENCE.

(2)TYPE T1 - NO SIGNIFICANT BREAKDOWN OF ORIGINAL PIECES.
TYPE T1S - SOFT, CAN BE BROKEN APART OR REMOLDED WITH FINGERS.
TYPE T1H - HARD, CANNOT BE BROKEN APART.
TYPE T2 - RETAINED PARTICLES CONSIST OF LARGE AND SMALL HARD PIECES.
TYPE T3 - RETAINED PARTICLES ARE ALL SMALL FRAGMENTS.

(3)USING NO. 10 SIEVE.

(4)CAN BE PERFORMED ON JAR-SLAKE TEST SAMPLES IF IN SITU NATURAL WATER CONTENT IS KNOWN. PI SENSITIVE TO DEGREE OF PULVERIZATION.

(5)REQUIRES SPECIAL PROCEDURES TO ASSURE GOOD DRAINAGE AND ADEQUATE COMPACTION (95% T-99) FOR LOOSE LIFT THICKNESS UP TO 24-IN. MAXIMUM.

Figure 12. Recommended durability index tests and suggested classification criteria for shales used in highway embankments

Volume 3. Different limiting values may be justified on the basis of local embankment performance experience. The descriptive categories for retained particles have been added to identify clay shales and especially western shales, which soften but sometimes do not degrade appreciably (T1S, Figure 12), and to identify shales that break down significantly but do not become smaller than the No. 10 sieve (T3, Figure 12). These types of shales could have a high slake-durability index that would be misleading without a descriptive supplement.

78. The intermediate classification for shales that degrade appreciably but are relatively hard is intended to call attention to shales that may be a problem and should be given special consideration. Compaction-degradation and point load strength as described by Wood et al. (1978)* could also be developed and used for these shales. Special procedures or provisions are usually needed to assure good drainage and adequate compaction of hard nondurable shales (such as 95 percent T-99 or specified types of compaction equipment and procedures with provisions for test pads, and limitation of loose lift thicknesses to 24-in. maximum).

79. The slake test (paragraph 72) could be used as an alternative to the slake-durability test since the slake index, I_S , is approximately equal to $100 - I_D$ (Chapman, 1975). However, more experience is required to verify the correlation of I_S (or I_D) with embankment performance. The main disadvantage of the slake test is that five cycles of drying and soaking require a minimum time of 5 days even when using a fast drying technique such as a microwave oven. With a microwave oven, the slake-durability test can be run in less than 2 hours.

80. For hard shales, the test water in the slake-durability and jar-slake test should be checked for pH. A pH less than six indicates an acid condition and the shale mineralogy should be checked for minerals that can cause chemical deterioration as described in Vol. 1. Chemical deterioration of hard shales in Virginia with $I_D > 90$ percent has been studied by Noble (1977)** in connection with settlement of bridge approach embankments on I-64 at Clifton Forge (Vol. 1). He used soaking in dilute solutions of concentrated sulfuric acid (18M) and distilled water as a classification test with a 25 percent solution being more reactive and giving the same ranking in degree of deterioration as the modified sulfate soundness test. Noble recommends that hard, dark-colored shales be checked for iron sulfide and chlorite as a clay mineral, since this combination can have great potential for rapid weathering. Shales with iron sulfide (such as pyrite), upon oxidation

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* Wood, L. E., Sisiliano, W. J., and Lovell, C. W., "Guidelines for Compacted Shale Embankments," Highway Focus, Vol. 10, No. 2, May 1978.

** Noble, D. F., "Accelerated Weathering of Tough Shales," Final Report VHTRC 78-R20, Virginia Highway and Transportation Research Council, Charlottesville, VA, Oct 1977.

and access to water, produce sulfuric acid, which dissolves the chlorite. Noble also recommends that methods of construction and treatment be used for these shales to impede rapid weathering. Spreading of lime during placement and compaction and/or preventing infiltration of water are possible solutions. The above tests should be considered in classifying hard shales contemplated for rockfill on important projects where long-term settlement must be kept to a minimum. In contrast to acid reaction, some shales in the Western U. S. have dispersive tendencies (Vol. 1) and may react adversely in alkaline water (high pH).

81. For shales with a high loss of material through the No. 10 screen in the slake-durability test and the slake test, the minus No. 10 material can be used for Atterberg limit tests, with little additional effort to disaggregate particles. For example, a shale with $I_D = 60$ percent provides 40 percent of minus No. 10 sieve-size material with a dry weight of about 200 grams, enough for the liquid limit and plastic limit tests. The limits can be used to determine the rate of slaking if the natural in situ water content is known. (Soaked water content can be measured after the first cycle of the slake-durability or slake test or from the jar-slake test.) The limits can also be used to estimate the long-term or residual strength from correlation by Townsend and Gilbert (1974).* Shales with $I_D < 60$ percent could produce 40 percent or more fines in the embankment, which could control the overall shear strength.

82. Point load tests, as described in Vol. 4, may be useful as a field index test, provided that a unique point load index can be established for different shales within a cut and the index correlated with slake-durability index. A series of tests on siltstone with $I_D = 96$ percent and shale with $I_D = 38$ percent, showed similar point load index (I_P) values (see Figure 31, Vol. 4). Thus, the usefulness of the point load test would have to be evaluated during design studies.

Shale Sample Selection and Testing

Sample selection

83. Selection of shale samples for testing should be done under the supervision of a geotechnical engineer. Representative unweathered cores or pieces from each different shale layer should be tested unless durability and compaction properties have been previously established. The type and amount of shale required for the different tests are given

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* Townsend, F. C. and Gilbert, P. A., "Engineering Properties of Clay Shales, Report 2, Residual Shear Strength and Classification Indexes of Clay Shales," Technical Report No. S-71-6, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS, Aug 1974.

in Table 5. Soil mantle and weathered shale should be treated as soil with normal soil tests assigned.

Testing

84. Index tests for shale durability evaluation can be performed on dried cores or chunks and natural water content determined on cores or chunks sealed upon sampling to preserve them at their in situ water content. Usually compaction tests would be performed on nondurable shales, since they would be compacted as soil in thin lifts. Procedures for index tests are described in Vol. 1 and 3 and by Chapman (1975).

Compaction tests

85. For coarse graded shales, the AASTO T-99 (Method D) compaction test can be used (except as noted below) on scalped material or a modeled gradation (Vol. 4). The general approach is to model the estimated field gradation using minus 3/4-in. material compacted in the 6-in.-diameter mold. An example of the modeled gradation for a hypothetical field gradation is shown in Figure 13. The major problem with shales is that drying induces slaking and softening of shale pieces on rewetting. The result is usually excessive degradation during compaction testing and unrealistically high dry densities.

86. Excessive degradation can be minimized by processing (crushing and sieving) the shale rapidly (preferably in a humid room) into required fractions (such as 3/4 to 3/8, 3/8 to No. 4, No. 4 to No. 40) and sealing these fractional amounts in heavy plastic bags or lard cans. Samples of the modeled gradation should also be reconstituted rapidly from the individual fractions (in a humid room, if possible) to prevent further drying. New material is required for each compacted sample. Water contents for compaction tests should be at the in situ water content (since this may be how the material is compacted in the field) and at two higher water contents corresponding to feasible water contents that can be achieved during field compaction.

Test results

87. An example of a standard means of presenting test results as used in Indiana is shown in Figure 14. All pertinent information is included on one sheet, and valuable additional information is supplied on geologic description, physical properties, and soil classification.

Special tests

88. Compression index. For large shale embankments where compression (or settlement of the embankment) is of major concern, a compression index test can be performed (as described in Vol. 4) on compacted samples by applying a surcharge load corresponding to the vertical pressure of one-half the embankment height. After equilibrium has been reached

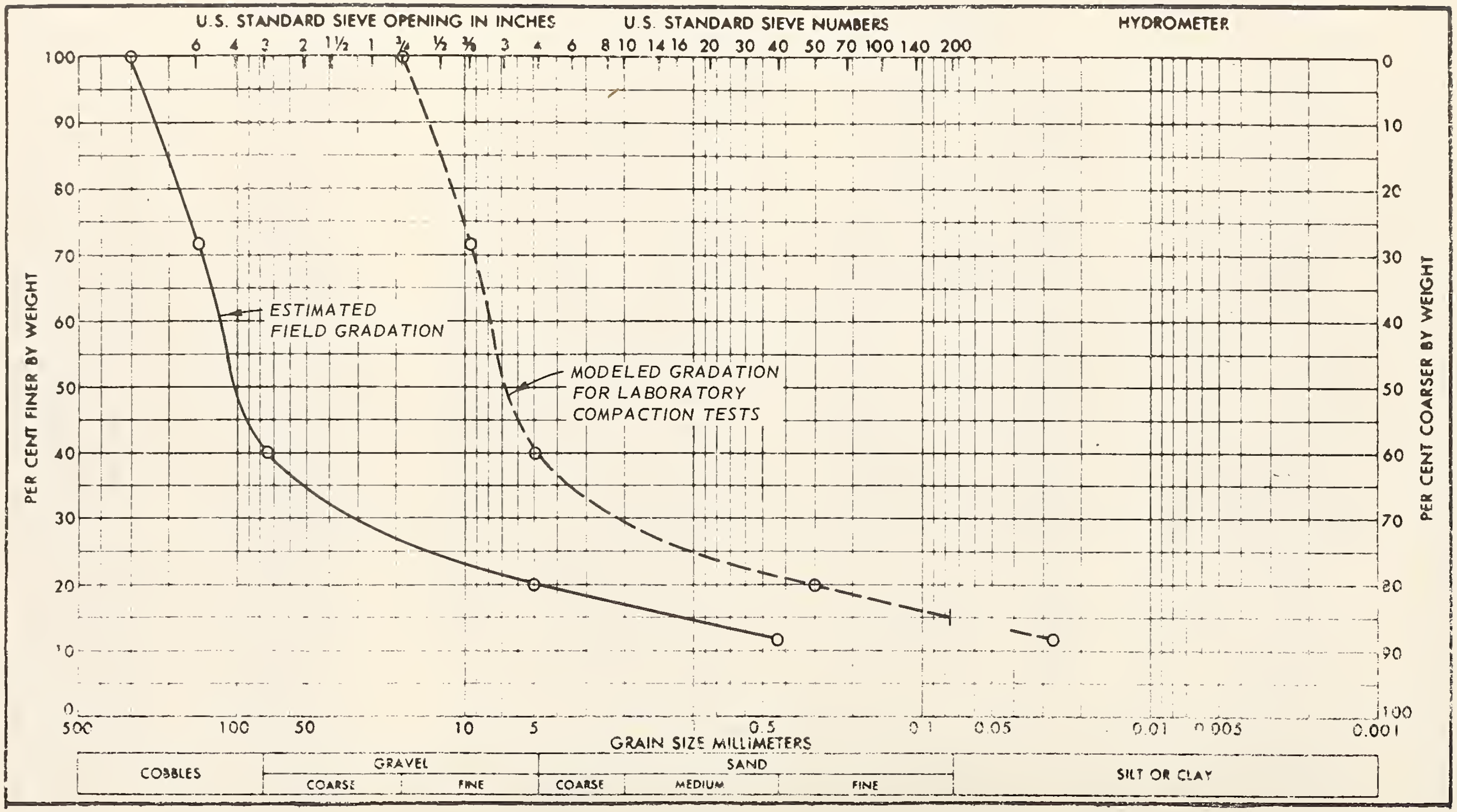

Figure 13. Example of proportional gradation used to model hypothetical field gradation

TD-445 (4-74)
1M 4-74

INDIANA STATE HIGHWAY COMMISSION
DIVISION OF MATERIALS AND TESTS

29 June 1977

Purchase Order No. ______

CONTRACT No. R-10783 PROJECT No. TQF-105-1(1) Const. ROAD No. S.R. 145

REPORT ON SAMPLE OF SHALE

Laboratory Number: 77-55441 County: Orange
Quad/Date: Greenbrier,Ind./1956 N1/2 SE ¼ SE ¼ SE ¼S 15 T 1S R 2W 2nd PM
Date Sampled: 05-26-77 Date Received: 05-26-77 Submitted by: C.A. Andrews
Station: 364+25 Offset: 50' Rt. Depth: 28' Elevation: 683' Sample marked: #4
Source of material: Cut Proposed use: Embankment

GEOLOGICAL DESCRIPTION B.P.U. Crawford Upland

System: Pennsylvanian Series: Pottsville Stage (Formation): Mansfield

TEST RESULTS

GENERAL PHYSICAL DESCRIPTION

Color: Dark gray Hardness: Soft [X] Medium [ ] Hard [ ]; Fissility: Massive [ ] Flaggy [X] Flaky [ ]

SHALE CLASSIFICATION

Soil like [X] Intermediate 2 [ ] 1 [ ] Rock like [ ]

Slaking Index:
Cycle No: (1) 34.0 (5) 76.7

Slake Durability Index	200 Rev.	500 Rev.
Dry:	43.7/26.7*	22.9
Soaked:	20.7	20.4

Fissility Number: 27.6
Modified Soundness Test: % Loss

PHYSICAL PROPERTIES

Natural Wet Density:	148.0	lbs/cuft.
Natural Dry Density:	135.4	lbs/cuft.
Natural Moisture:	9.3	percent
Specific Gravity:	2.735	
Ph:	6.3	
Shrinkage Limit:	15.2	%
Lineal Shrinkage:	6.7	%
Loss on Ignition:	7.4	%

SOIL CLASSIFICATION

Textural:	Silty Clay (Shale)
AASHTO:	A-6(15)
Plastic Limit:	24.1 %
Liquid Limit:	38.8 %
Plasticity Index:	14.7 %

% Sand: 6.1 Silt: 50.4 Clay: 26.6 Colloids: 16.9
Clay is defined as that material smaller than 0.005 mm in diameter.

MOISTURE DENSITY RELATIONS

(Note: Minus [X] No. 4 [ ] ¾ inch material)

Maximum Wet Density:	130.3	lbs/cuft.
Maximum Dry Density:	113.0	lbs/cuft.
Optimum Moisture:	15.0	percent

CALIFORNIA BEARING RATIO

As Compacted CBR Value:	9.2	%
After Soaking CBR Value:	1.7	%
Average % Swell:	2.0	%

(CBR value at 95% of Maximum Dry Density)

REMARKS: *Second Cycle, 200 Rev. Dry Durability Value as per JHRP Report 75-11 by D. R. Chapman (pg. 34)
It is considered that this material is not suitable for use within two (2) feet of subgrade elevation.

RR/vf

E-1-1

Figure 14. Example of shale properties summarized in a standard form (courtesy of Indiana State Highway Commission)

under this load (usually in 1 day), the sample can be seepage-saturated by introducing water under a head of about 4 ft (arbitrarily selected standard). Percent compression upon saturation (4 to 10 days) for samples at different densities can give a relative measure of expected field performance. This is illustrated in Figure 15 for four different shales (Vol. 4) using a 1000-lb surcharge load (vertical stress of 35 psi which is equivalent to 40 ft of embankment height at 127 pcf density) and the percent compression correlates roughly with slake-durability index. The effect of increasing compacted dry densities by 2 to 4 pcf on reducing soaked compression and corresponding settlement is evident from the data shown in Figure 15.

89. Permeability. Relative permeabilities can also be measured after the compacted samples are soaked and seepage water has emerged above the sample surface. A falling head-type test is easy to perform and can be completed in one or two days (see Vol. 4). The permeability can provide an estimate of seepage rates and how rapidly similarly compacted material can drain or become saturated under surface or groundwater infiltration.

Gradation tests

90. Gradation tests on compacted samples are not recommended for routine use. The process of breaking the sample apart and sieving causes further degradation, and the results can be entirely misleading. Special procedures for minimizing degradation during gradation tests include the following:

a. Gently pulling compacted sample apart in humid room to prevent drying and running wet sieve analyses (moist shale does not tend to slake significantly when soaked).

b. Samples not densely compacted can be air-dried until they become friable, then broken apart by hand and sieved as described by Sisiliano et al. (1978).*

Storage and retrieval of data

91. Consideration should be given to a computerized storage and retrieval system for geologic and laboratory test data on shales as a developing source of information for future projects. The advantages include minimizing testing for a particular shale member and the potential for correlating index properties with shear strength, soaked compression, excavation characteristics, and service performance. A

* Sisiliano, W. J. et al., "Report of a Shale Test Pad, R-Contract No. 10783, TQF-Project No. 105-1(1) Const., S.R. 145 in Orange County," Indiana State Highway Commission Division of Materials and Tests, Soils Department, Indianapolis, IN, May 1978.

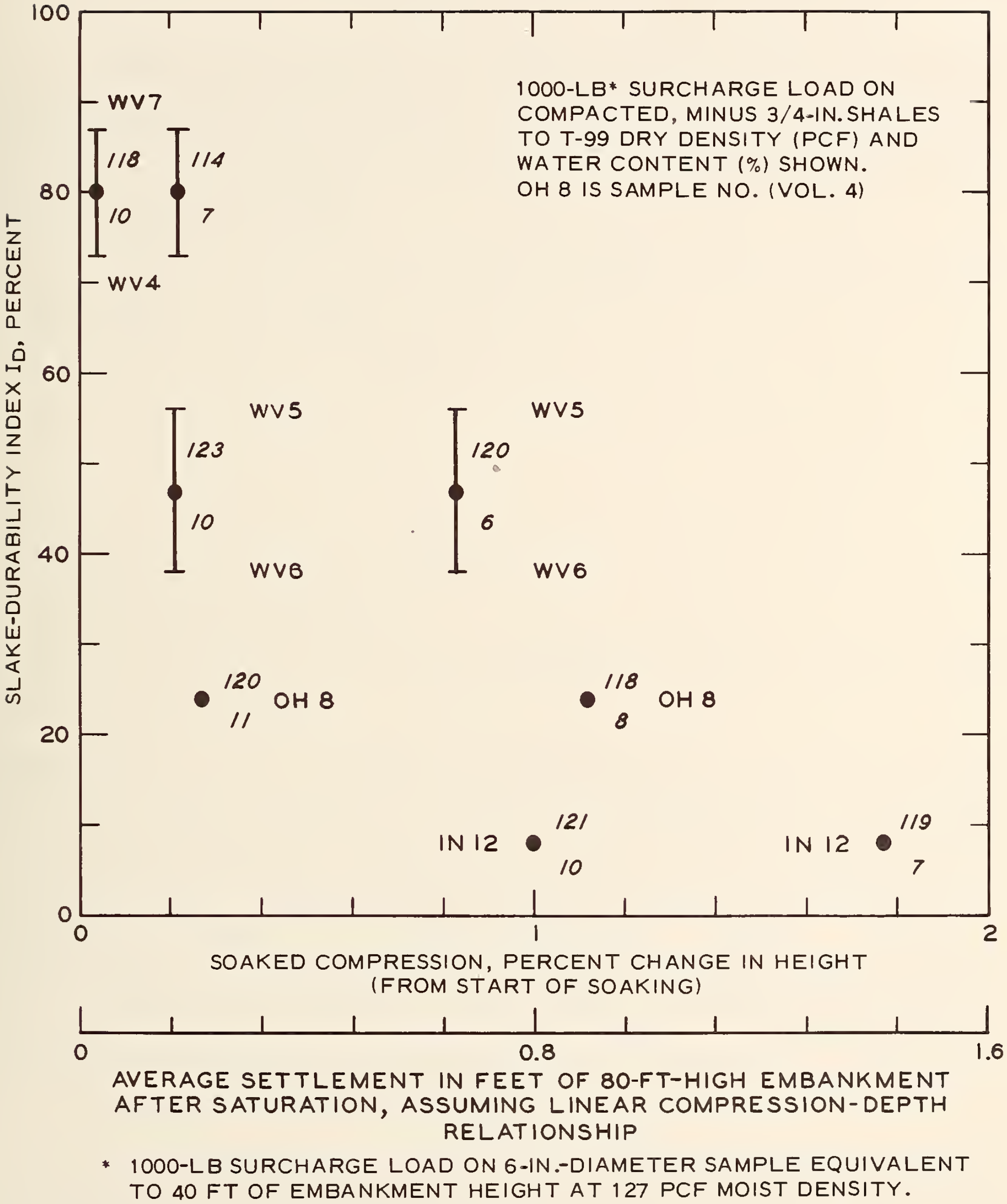

Figure 15. Soaked compression of minus 3/4-in. compacted shale samples related to slake-durability index

preliminary correlation of slake-durability and lift thickness is described in Vol. 3. Use of a retrieval and storage system in Indiana is discussed by van Zyl (1977).*

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* van Zyl, Dirk J. A., "Storage, Retrieval and Statistical Analysis of Indiana Shale Data," Joint Highway Research Project, JHRP-77-11, Purdue University, Jul 1977.

PART V: SHALE FORMATION EXCAVATION CHARACTERISTICS

Influence on Gradation

92. Excavated materials from shale formations used in embankments must have the proper gradation (Part II) to insure adequate compaction of nondurable shales in thin lifts or durable shales in thicker, rockfill lifts. Special provisions on ripping and/or blasting procedures to tailor these methods to the formation features may be necessary to obtain the required gradation.

93. Important features of shale formations that control the maximum size and gradation range include:

- a. Depths of weathering.
- b. Joint spacing.
- c. Degree of fracturing.
- d. Bedding plane spacing.
- e. Shale hardness.
- f. Attitude, thickness, and spacing of shale beds and harder rock layers.

Alternating thin beds of shale and limestone, which can be ripped using a heavy tractor with a single-tooth ripper, produce slabs of limestone that require extra tractor coverages to break them down. Thick layers of hard, massive shale or claystone and siltstone that require blasting can produce large blocks, several feet in size (depending on joint spacing) if large diameter, widely spaced holes are used. Thus the minimum weight and type of ripping equipment and amount of ripping and tracking might need to be specified in the first case, while the maximum hole diameter, spacing, and other details of blasting procedures might need to be specified in the second case. In addition, further breakdown may be required during placement, using tractor coverages, heavy disking, and compaction equipment.

94. The alternative to specifying ripping and blasting restrictions and extra working during placement is to specify the required gradation range of the excavated materials prior to compaction. This approach may be successful if the contractor has all available information on the formation features and a proven record of achieving the required results. Both approaches require an adequate inspection staff to enforce the control provisions and frequent geotechnical staff visits to solve unforeseen technical problems during construction.

95. *The main advantage in establishing the required gradation for shale embankment materials during the design stage is that the sample requirements, laboratory testing feasibility (versus field test pads at the start of construction), and methods for selecting design parameters are also established in a logical manner.*

Rippability Assessment

Methods

96. Rippability of shale formations can be roughly estimated from seismic velocity and past experience. Experience from a number of projects by Church (1972)* is summarized in Figure 16 for sedimentary deposits with three different layer-thickness categories. Three weathering categories were developed based on a number of field factors and measured seismic velocity data to provide the charts shown in Figure 16. The weathering category can be used to assess the rippability of various depths, as shown by the example on this figure.

97. Another rippability rating method refined by Weaver (1975)** is based on seismic velocity, rock hardness, weathering, jointing characteristics, and bedding orientation and is summarized in Table 6. Numerical ratings for each variable are summed to obtain a total rating related to the difficulty of ripping. Seismic velocity and joint spacing are the major variables, followed in order by strike and dip orientation, weathering, and rock hardness. This chart requires more detailed geotechnical information than the evaluation method by Church and could be modified based on local area experience.

98. A third method of assessing excavation difficulty has been suggested by Royster (1976).† The classification scheme is based on an index or indicator number from one to ten, as shown in Table 7, for various degrees of excavation difficulty. This system would have to be developed for local formation conditions based on accumulated experience. As suggested by Royster, the designations could be included on profiles and sections showing subsurface conditions (Part III) to provide construction-oriented information, as shown in Figure 17.

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* Church, H. K., "433 Seismic Excavation Studies: What They Tell About Rippability," Roads and Streets, Jan 1972.

** Weaver, J. M., "Geological Factors Significant in the Assessment of Rippability," The Civil Engineer in South Africa, Dec 1972.

† Royster, D. L., "Designation of Excavation Characteristics for Materials Identified in Field Investigations," Transportation Research Record 612, Transportation Research Board, Washington, D. C., 1976.

FIELO FACTORS ANO WEATHERING CATEGORY			
FIELD FACTOR	WEATHERING CATEGORY		
	MINIMUM	AVERAGE	MAXIMUM
RAINFALL, AVG ANNUAL	TO 1S	1S" TO S0"°	ABOVE S0
TEMP AVG ANNUAL	BELOW 40	40 TO 80 °	ABOVE 80
HUMIOITY AVG ANNUAL	BELOW 40%	40 TO 80%°	ABOVE 80%
TOPOGRAPHY	MOUNTAINS°	HILLS	VALLEYS
ROCK EXPOSURE	SOUTH	EAST-WEST°	NORTH
VEGETATION	SPARSE	NORMAL°	ABUNOANT
RESIOUALS	THIN°	MEDIUM	THICK
ROCK OUTCROPS	HEAVY°	LIGHT	NONE
ROCK GRAIN	SMALL	MEOIUM°	LARGE
ROCK COLOR	OARK°	MEOIUM	LIGHT
ROCK OURABILITY	HARO	MEOIUM°	SOFT

° EXAMPLE. OVERALL CATEGORY OF "AVERAGE" SHOWS THE FOLLOWING RESULTS FOR THE GIVEN CONOITIONS:

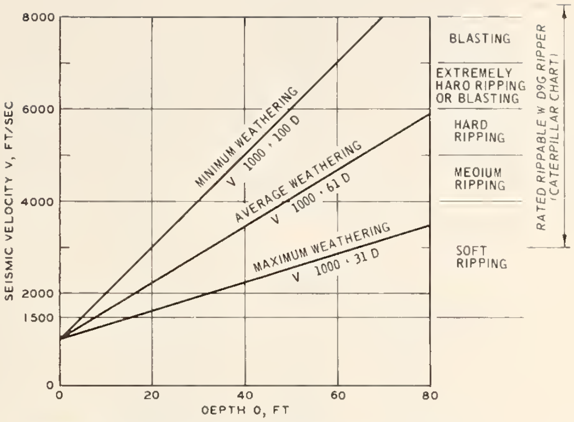
a. SHALES ANO SILTSTONES <1-FT THICK BEOS, 20 TO 30 FT SOFT RIPPING

b. SILTSTONES ANO SANOSTONES. >3-FT BEOS, 30 TO 40 FT MEOIUM TO HARO RIPPING

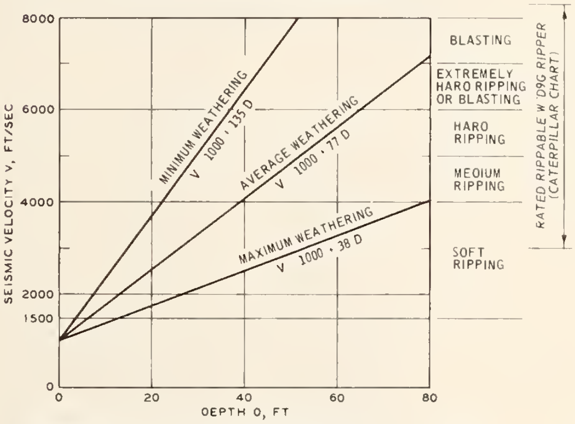
c. SHALES AND SILTSTONES, <1-FT THICK, 40 TO S0 FT SOFT RIPPING

d. SANOSTONE >3-FT THICK BEOS, S0 TO 60 FT EXTREMELY HARO RIPPING OR BLASTING TO 57 FT, BLASTING TO 60 FT

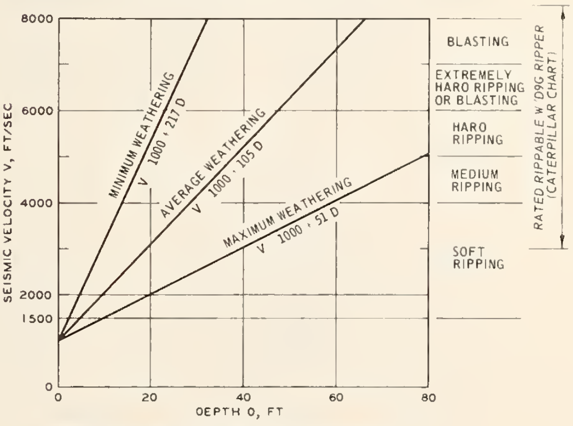
NOTE SEISMIC VELOCITY VERSUS OEPTH CHARTS OEVELOPEO BY PLOTTING AVERAGE VALUE FOR EACH PROJECT, OIVIOING RANGE OF OATA INTO THREE SECTORS, ANO ORAWING LINE THROUGH CENTER OF SECTOR (CHURCH 1972)



a. SEDIMENT. ROCK, LESS THAN 1-FT THICK BEDS SUCH AS SHALES, SILTSTONES, SANDSTONES (26 PROJECTS)



b. SEDIMENT. ROCKS, 1 TO 3-FT THICK BEDS, SUCH AS SILTSTONES AND SANDSTONES (6S PROJECTS)



c. SEDIMENT. ROCKS, MORE THAN 3-FT THICK BEDS, SUCH AS SANDSTONES AND LIMESTONES (87 PROJECTS)

CHURCH, HORACE K., "433 SEISMIC EXCAVATION STUOIES: WHAT THEY TELL ABOUT RIPPABILITY" ROAOS ANO STREETS, JANUARY 1972.

Figure 16. Rippability evaluation based on field experience (from Church, 1972)

Table 6. Rippability Rating Chart (Modified after Weaver 1975)

Rock Class: Description: Seismic Velocity: Rating:	I Very Good >2150 m/s (>7050 ft/sec) 26	II Good 2150-1850 m/s (7050-6070 ft/s) 24	III Fair 1850-1500 m/s (6070-4920 ft/s) 20	IV Poor 1500-1200 m/s (4920-3940 ft/s) 12	V Very Poor 1200-450 m/s (3940-1480 ft/s) 5
Rock Hardness: Description: Rating:	Extremely Hard Requires many blows with geologic pick to break through intact material; rings under hammer blow. 10	Very Hard Hand specimen breaks after more than one blow with pick; rings under hammer blow 5	Hard Cannot be scraped with a knife; hand specimen breaks under one firm blow with pick; rings under hammer blow. 2	Soft Can just be scraped with knife; firm blows with pick point cause 1-mm to 3-mm indentations; dull sound under hammer. 1	Very Soft Can be peeled with knife; crumbles under firm blows with geologic pick point pieces up to 3-mm size can be broken by finger press. 0
Rock Weathering: Description: Rating:	Unweathered Discontinuities usually tight; no discoloration 9	Slightly Weathered Discontinuities may be open; surfaces slightly discolored. 7	Weathered Rock discolored, discontinuities may be open with discolored surfaces and alteration starting to penetrate inward. 5	Highly Weathered Rock mass discolored, discontinuities may be open and surfaces discolored; alteration penetrates deeply but core stones present. 3	Completely Weathered Entire mass discolored and changed to soil but original fabric mainly preserved; occasional small corestones present. 1
Joint Spacing: Rating:	>3000 mm (>120 in.) 30	3000-1000 mm (120-40 in.) 25	1000-300 mm (40-12 in.) 20	300-50 mm (12-2 in.) 10	<50 mm (<2 in.) 5
Joint Continuity: Rating:	Non-continuous 5	Slightly continuous 5	Continuous - no gouge 3	Continuous - some gouge 0	Continuous with gouge 0
Joint Opening-Gouge: Rating:	No separation 5	Slight separation 5	Separation <1 mm (0.04 in.) 4	Gouge - <5 mm (0.2 in.) 3	Gouge - >5 mm (>0.2 in.) 1
Strike & Dip Orientation: Rating:	Very unfavorable 15	Unfavorable 13	Slightly unfavorable 10	Favorable 5	Very favorable 3
Total Rating: Rippability Assessment: Tractor Type: Horsepower:	100-90 Blasting Required	90-70 * Blasting & extremely hard ripping DD96/D9G 780/385	70-50 Very hard ripping D9/D8 385/270	50-25 Hard ripping D8/D7 270/180	< 25 Easy ripping D7 180

* Ratings in excess of 75 should be regarded as unrippable without preblasting.

Table 7. Example of Excavation Characteristic Designations (ECD) (Royster, 1976)

Excavation Index (ECD)	Degree of Excavation Difficulty	Examples	Excavation Index (ECD)	Degree of Excavation Difficulty	Examples
1	May be easily scraped	Relatively dry sand or silt; some clays	6	May be ripped with some difficulty	Very slightly weathered shale; thin and slabby, disjointed limestones and siltstones; saprolite ("rotten" igneous or metamorphic materials)
2	May be scraped or bladed	Moist gravel, sand or silt; most clays	7	Rippable with great difficulty	Thin-bedded chert with clay seams; thin-bedded limestone or siltstone with interbedded shale.
3	May be easily bladed or may be scraped with difficulty	Moist clay with minor--less than 25%--small disseminated rock particles, some highly weathered shales, some organic materials	8	Requires blasting (up to 25%)	Weathered granite, slate, and other igneous or metamorphic rocks; friable sandstone; medium- to thick-bedded limestone with cutters, or disjointed with clay or shale seams; soils with rock pinnacles or large boulders
4	May be bladed with difficulty	Clay with moderate to heavy--25% to 50%--small disseminated rock particles, or with minor pinnacle and/or boulder content; some moderately weathered shales; colluvium with minor boulder content; sanitary landfill material; alluvial boulders	9	Requires blasting (25 to 50%)	Hard shale; thin- to medium-bedded sandstone, siltstone or limestone with interbedded shale, soils with numerous rock pinnacles or large boulders
5	May be easily ripped, dredged or draglined, or may be bladed with great difficulty	Clay with heavy disseminated to some bedded chert; slightly weathered shale; talus or colluvium with heavy boulder content; saturated clay, silt, sand or gravel	10	Requires blasting (greater than 50%)	Thin- to thick-bedded sandstone, siltstone, and limestone; granite, slate and other well-indurated or "fresh" igneous metamorphic, and sedimentary rocks

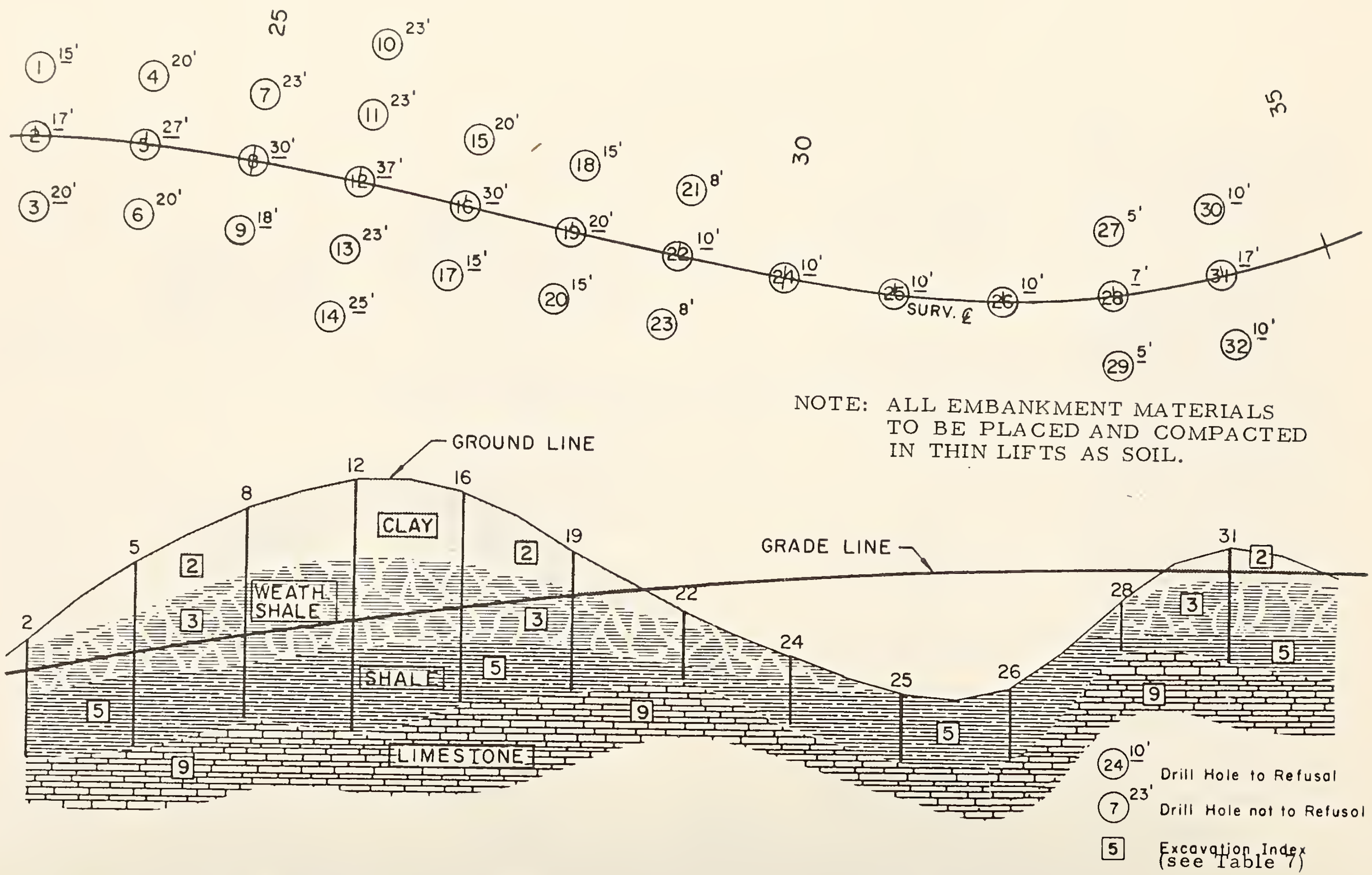


Figure 17. Schematic of boring pattern in plan and profile showing excavation characteristic designators (ECD) (Royster, 1976)

Gradation

99. Considerable experience and judgment are required in estimating gradation ranges for shales excavated by ripping. Bedding thickness, joint spacing, and degree of fracturing are the controlling variables. A D-9 tractor with a three-tooth ripper will produce better fragmentation to a depth of about 20 in. than with a single-tooth ripper. For difficult ripping, the two outer ripper teeth can be used with overlapping coverage or cross-ripping. The use of minimum requirements will not unduly restrict the contractor's options.

Controlled Blasting

100. The use of time delay patterns in blasting can increase fragmentation. However, maximum rock sizes still may be too large for direct use in compacted shale embankments, as illustrated by experimental blasting tests in 1973 at the R. D. Bailey Project, West Virginia (Bechtell, 1975).* The tests were made in shale and sandstone strata of the Kanawha formation and include extensive gradation measurements. The results, summarized in Table 8, indicate that even with time delay blasting and closely spaced, small-diameter holes (Tests PB-4A and PB-4B), maximum rock size ranged from 2 to 8 ft, with about 20 to 40 percent larger than 10 in. While this gradation would be suitable as rockfill if the shale were durable, it would not be suitable for thin lifts if the shale were nondurable. Without the time delay, even larger rock sizes were produced (Test PB-4C).

101. Small-scale tests in dolomite by Ash and Smith (1976)** showed that ratios of $L/B = 3$ and $S/B = 1$ (see nomenclature, Figure 18) used with time delay (instead of instantaneous) firing also increased fragmentation. Increasing the bench height produced increased rupture by column bending. In a cement quarry operation, fragmentation of cemented conglomerate was improved for loading using a front-end loader by reducing the burden, B, by 1 ft and increasing the bench height, L, to 26 ft. Cost effectiveness was maintained by increasing the spacing by 4 ft. The final L/B was 2.2 and S/B was 1.3. The type of explosive used is also important; slow-acting explosives, rich in gas, may be an advantage in jointed rock. General guidance on

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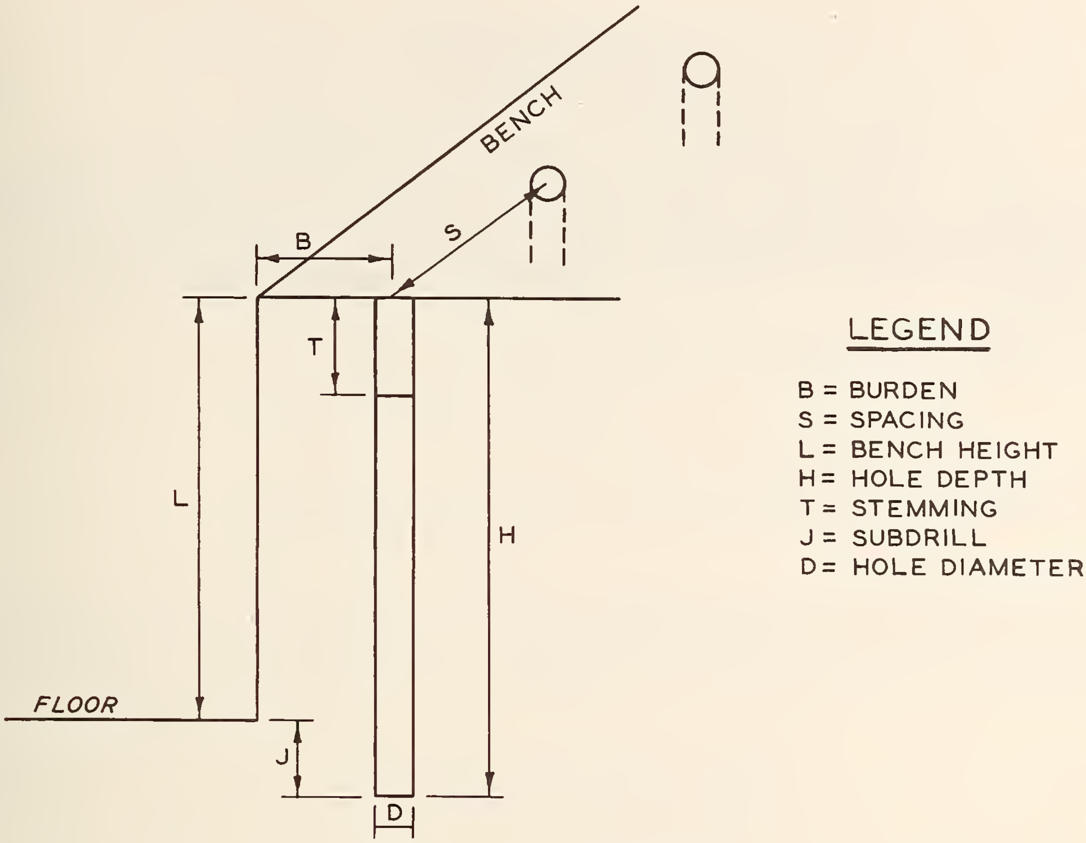
* Bechtell, W. R., "Project R. D. Bailey Experimental Excavation Program," Technical Report No. E-72-2, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS, Jun 1975.

** Ash, R. L. and Smith, N. R., "Changing Borehole Length to Improve Breakage: A Case History," <u>Second Conference on Explosive and Blasting Techniques</u>, Society of Explosives Engineers, Louisville, KY, 28-30 Jan 1976.

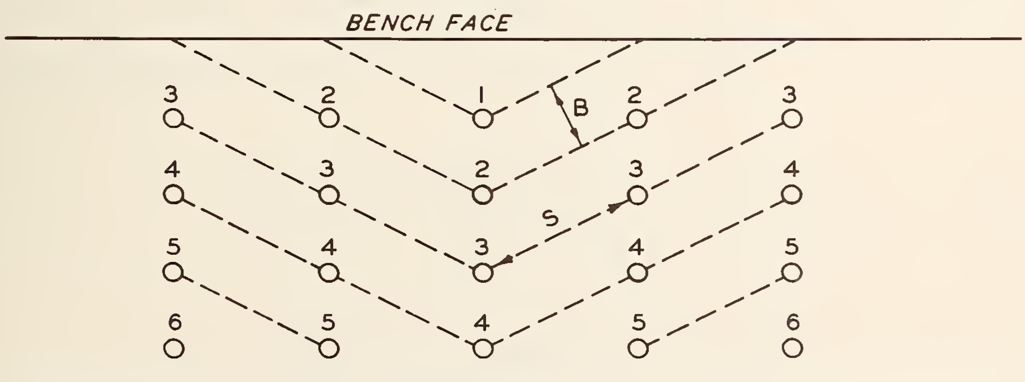
Table 8. R. D. Bailey Project, Experimental Excavation Tests
(Producing Greatest Fragmentation) (Bechtell, 1975)

Test No.	Material	Hole Diam. in.	Avg. Depth ft	Hole Pattern	Explosive Type and Powder Factor	Firing Delay Sequence	Burden ft	Spacing ft	Stemming ft	Sub-drill ft	Pre-split Holes	Buffer Holes	Avg. Frag. Depth ft	Max. Size ft	Plus 10-in. %	Minus 2-in. %
PB-3	Weathered shale	6-3/4	15	19-ft square	ANFO 0.79 lb/yd^3	None; breaking to horz. surface	NA	NA	6	3	yes	yes	15	1 to 5	3 to 15	30 to 48
	(Free faces on 3 sides)									(Easily ripped using D-8)						
PB-4	Weathered shale, mod. hard shale, coal seams, & sandstone	9	43	24-ft square	ANFO 0.82 lb/yd^3	"V" cut, 25 msec delays from free face	17	34	14	3	yes	yes	34	3 to 7	23 to 27	12 to 17
										(Ripped to a depth of 16 ft)						
PB-4A*	Sandstone & hard shale	3	6	10 by 5 ft staggered	ANFO 1.12 lb/yd^3	8 rows, parallel 25 msec	5	10	2	0	NA	NA	6	2 to 8	22 to 38	3 to 12
PB-4B*	Sandstone & hard shale	3	9	7-ft square	ANFO 0.9 lb/yd^3	8 rows, zig zag 25 msec	5	10	4	0	NA	NA	9	2 to 8	22 to 40	0 to 10
PB-4C*	Sandstone & hard shale	3	12	5 by 5 ft staggered	ANFO 2.03 lb/yd^3	None	NA	NA	4	0	NA	NA	<12	4 to 9	40 to 60	0 to 5
PB-4E*	Sandstone	3	6	5 by 5 ft staggered	DOW MS-80-25 Al-AN slurry 2.3 lb/yd^3	7 rows; row 1 to 3, 8 msec; row 4 & 5 50 msec row 6 & 7 75 msec	5	5	3	0	NA	NA	6	3 to 7	23 to 40	11 to 24

* To complete the excavation depth for test PB-4.



a. Basic nomenclature



b. Chevron or "V" delay pattern (numbers indicate order of delay sequence)

Figure 18. Blast hole and pattern nomenclature

good blasting practices to achieve desired breakage is given by Konya (1977).*

Practical Solution

102. *The best approach in achieving required gradation ranges would be a cooperative effort with the contractor to determine optimum procedures for breaking down shales and rock at the start of the project (and in subsequent, different shale formations). This effort would allow adjustments in ripping and blasting methods to obtain the desired gradation.*

* Konya, C. J., "Good Blasting Practices Mean Money in the Bank," Rock Products, Nov 1977.

PART VI: SHALE EMBANKMENT DESIGN

103. The design of shale embankments involves four main steps:

- a. Assessment of potential problems with shale materials, considering geologic conditions, shale durability, and construction practices.
- b. Selection of appropriate design features, material properties, and construction procedures to meet desired settlement and stability criteria.
- c. Preparation of plans and specifications, including special provisions and construction control techniques to achieve design criteria.
- d. Development of an appropriate subsurface and/or surface instrumentation plan for monitoring the performance of major embankments.

Because of the variability in shale formations and shale durability, special design and construction consideration for shale embankments are necessary to achieve adequate compaction and prevent harmful saturation of embankment materials.

Potential Problem Assessment

104. A realistic assessment of potential problems (other than poor foundations) with embankments constructed in shale formations requires a thorough understanding of the causes of distress and the role of contributing factors, such as those listed in Table 9 and discussed further below.

- a. While inadequate compaction, saturation, and shale deterioration are the primary causes of large settlements and slides, the contributing factors may involve one or several of the following, as discussed in Vol. 1 and 4:
  - (1) Inadequate foundation benching and drainage along slidehill locations and on transverse slopes beneath the fill-to-cut transition.
  - (2) Difficulties in breaking down hard shale and rock materials, such as interbedded shale and limestone during excavation and placement.
  - (3) Uncontrolled mixing of soil, shale, and rock in the same lift during placement.

Table 9. Summary of Factors to Consider in Assessment of Potential Problems with Shale Embankments

Important Factors	Potential Problem Category: Minor	Moderate	Major	Factor Influence
Formation Geology in Cuts	(Examples only; should be revised on basis of local experience and judgment)			
Depth of Soil Cover; Depth of Weathering	Thin	Shallow ($\leq$ 5 ft)	Deep (> 5 ft) or highly variable	Depth of sidehill benching and need for selective placement (or grading)
Thickness of shale and harder rock layers	$\geq$ 5 ft	3 to 5 ft	< 1 to 3 ft	Feasibility of selective excavation and placement
Interbedding of shale with harder rock	Thick beds	Few thin beds	Alternate thin beds	Selective excavation and compaction difficulties
Dip of bedding at cut-fill transition and along sidehill locations	Horizontal	Slight dip (< 5 deg) into embankment	Steep dip (> 10 deg) into embankment	Potential seepage into embankment
Joint spacing	Wide (> 10 ft)	Medium (5 to 10 ft)	Close (1 to 5 ft)	Maximum size of excavated rock
Groundwater levels and seepage from springs and through rock layers during wet season	Low	Moderate	High	Potential saturation of shale
Excavation Characteristics	Easily ripped	Difficult to rip	Very difficult ripping and/or blasting required	Maximum size and gradation of excavated materials
Shale Durability				
Degree of deterioration	Low	Medium	High	
Type of deterioration	Few cracks	Hard chunks	Slakes into silt or clay	Lift thickness and compaction requirements
Classification	Durable	Durable	Nondurable	
Type of Embankment	Cross Valley (or Through)	Skew or combination	Sidehill	Benching and foundation drainage requirements
Current Design & Construction Practices				
Slope benching and drainage; Rock Drainage Blankets; Underdrains and spring drains; special excavation procedures or selective excavation and placement	Routinely specified with required details and locations shown on plans	→	As required during construction	Preventing saturation of embankment materials; Proper utilization of shale, rock, and soil
Mixing of shale, soil, and rock	Controlled to prevent	→	Uncontrolled	Undesirable mixing of nondurable shale and soil with rock
Field classification of shales suitable for rockfill	Successful or not used as rockfill	→	Unsuccessful	Undesirable mixing of nondurable shale and soil with rock
Lift thickness; Compaction	Specified and well controlled during construction	→	Specified but not well controlled; No special requirements	Difficulties in achieving adequate compaction
Embankment Performance Experience				
Settlement (excluding bridge abutments)	< 6 in.	6 to 12 in.	> 12 in. continues for several years	Cost for maintenance or remedial treatment or reconstruction
Slope stability	Minor sloughing	Shallow slides	Large slides encroaching into roadway	Cost for maintenance, remedial treatment, or reconstruction

(4) Lack of reliable tests and criteria for distinguishing durable shales from nondurable shales.

(5) Use of excessive lift thickness.

(6) Lack of specific compaction requirements and procedures.

b. Although a number of problem shale formations have been identified (Vol. 1), the relative influence of construction procedures and the relative importance of other factors listed in Table 9 that contribute to inadequate compaction, saturation, and shale deterioration is not well defined. Consequently, considerable judgment is required in assessing potential problems.

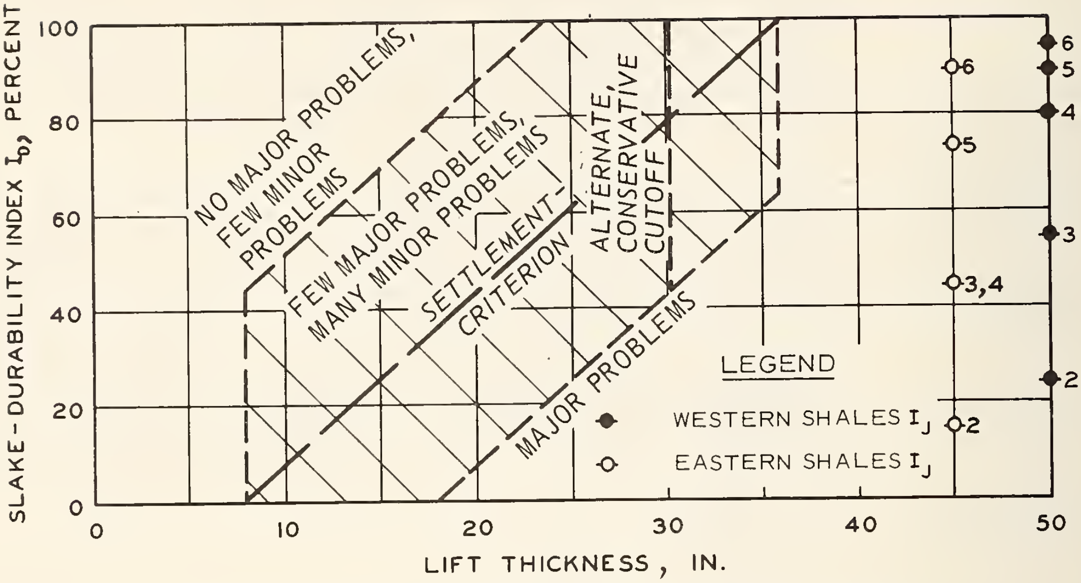
c. A valuable guide can be established if slake-durability index, lift thickness, compaction procedures, and performance data are collected and correlated. An example of a preliminary correlation of slake-durability index and lift thickness described in Vol. 3 is summarized in Figure 19a. The criterion in Figure 19a was established using performance experience (mainly settlement) for 83 embankments in 15 States. The examples in Figure 19b illustrate that the choice of lift thickness (points A-D or E and F) depends on the consequences of post construction problems and acceptable maintenance costs.

d. Considerable experience with shale construction projects and a good knowledge of formation geology and excavation characteristics are required to determine, for example, whether normal construction practices will cause undesirable mixing of nondurable shale and soil with rock because of variable stratigraphy, as discussed in Part II and lead to unacceptable settlements. Valuable information on specific projects and regional conditions is contained in State Highway Agency internal reports on slide investigations and repairs and in research reports and published papers by state geotechnical engineers (see references in this manual and bibliography in Vol. 1 and 4). The greater the detail on geologic conditions and shale durability developed during the project field investigation, the less will be the degree of conservatism required in assessing potential problems and the need for extensive use of special measures.

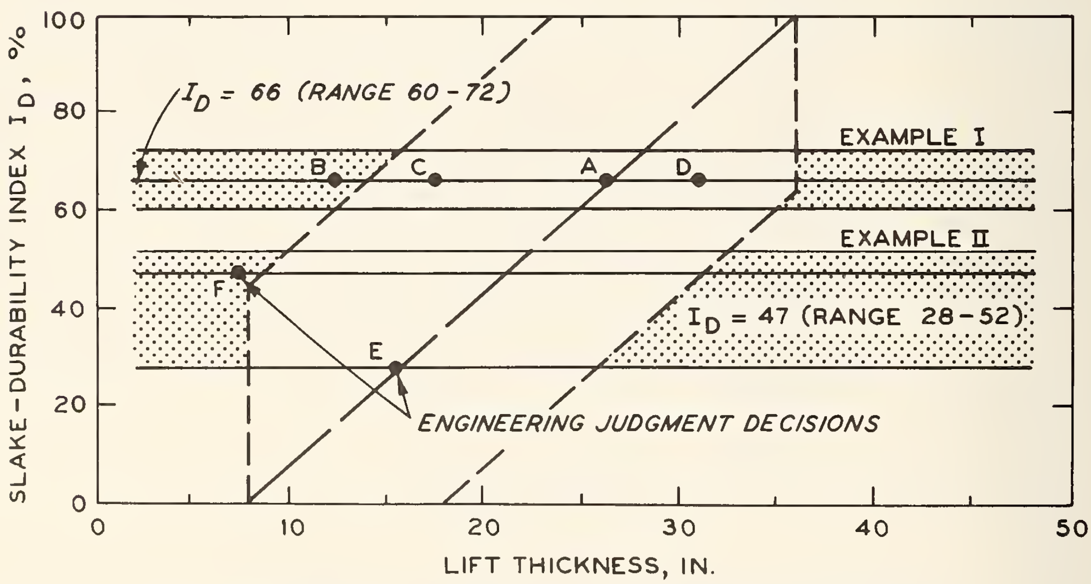
Design Features

Types

105. Normal design features required for shale embankments include foundation benching and drainage, material usage (by specifying excavation and placement procedures), compaction requirements, and slope design. Special design features for problem locations--such as deep colluvium, narrow right-of-way areas where high embankments require steeper than normal slopes, and areas of excessive seepage--include the following:



a. Preliminary criterion for evaluating embankment construction on the basis of slaking behavior



b. Hypothetical examples of determining suitable lift thickness from test results

Figure 19. Preliminary lift thickness criterion based on slake-durability index and settlement performance data

a. Berms.

b. Shear trenches with underdrains.

c. Retaining structures.

   (1) Rock buttresses.

   (2) Reinforced earthwalls.

   (3) Gabion walls.

   (4) Crib walls.

d. Drainage measures.

   (1) Longitudinal and transverse underdrains.

   (2) Rock drainage blankets (or pads).

   (3) Horizontal drains.

   (4) Vertical wells.

Selection

106. The type and extent of the design measures selected depends on the amount of information on shale formation geology and variability, excavation characteristics, shale durability and variability, and groundwater conditions. The selection also is influenced by settlement and stability criteria, anticipated construction problems, and the allowable degree of risk in relation to the long-term performance for the type of highway. Incomplete field information coupled with stringent requirements for small settlements and long-term stability of slopes may dictate selection of all feasible design measures and result in overdesign.

Foundation benching and drainage

107. Foundation benching and drainage as discussed below are particularly important for shale embankments to prevent saturation and softening of nondurable shales and soil at the base of the embankment.

a. For sidehill slopes generally steeper than 6:1 and high embankments (usually higher than 50 ft) on transverse slopes, foundation benching is necessary to allow compaction of initial fill layers on a level surface and to provide greater stability. The minimum bench width should correspond to the width of excavation equipment (usually scrapers with a width of 12 to 15 ft).

_b_. In deep residual soil, a rock drainage blanket (or pad) should be designed to intercept and drain subsurface seepage blocked by the embankment. Where rock is scarce or selective grading is not feasible, underdrains should be designed to provide drainage at the embankment base. Old landslide areas, unstable colluvium or areas of high subsurface seepage may require excavation of unstable material, benching, rock drainage blankets, underdrains, horizontal drains, and/or vertical drains to provide a stable foundation and to prevent saturation of the embankment.

_c_. The use of filter fabric in underdrains, spring drains, trench drains, and interceptor drains can reduce the need for and cost of graded filter materials. Current uses by several state highway agencies are described in the May 1977 and May 1978 issues of "Highway Focus." Sample guide specifications on filter fabrics are contained in FHWA-TS-78-211, "Sample Specifications for Engineering Fabrics," Implementation Division (HDV-22), 1978, and Corps of Engineers Guide Specification, "Plastic Filter Fabric," CW 02215, November 1977. The Corps of Engineers Guide Specifications (CW 02215) supersedes the extracts contained in FHWA-TS-78-211. Comprehensive information on properties of filter fabrics and current state of practice has been reported by Steward et al. (1977).*

108. Typical or standard drawings for benching and a drainage blanket, such as shown in Figure 20, should be tailored to existing conditions. Trench drains are often used in lieu of a drainage blanket. These drains may be difficult to excavate in hard shale, limestone, and sandstone strata. An alternate method of placing select aggregate or gravel and perforated pipe at the toe of the bench backslope should be considered (Figure 21). Collector pipes should not be extended through the embankment, since they could be broken by settlement or deformation of the embankment. Groundwater seepage into the embankment (illustrated in Figures 4 and 8) is controlled by bedding stratification and attitude. The important influence of bedding attitude on bench drainage design is illustrated in Figure 21. The specific locations of geologic bedding and required drains should be shown on cross-section drawings in the construction plans for projects where the risk of settlement or slides must be minimized. Without specific information on the location and inclination of seepage strata, a general requirement for standard benching with drains** could be an unnecessary extra cost if strata dipped

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* Steward, J. E. et al., "Guidelines for Use of Fabrics in Construction and Maintenance of Low Volume Roads," USDA, Forest Service, Portland, OR, Jun 1977. (Reprinted as Report No. FHWA-TS-78-205.)

** Transportation Research Board, "Construction of Embankments," Synthesis of Highway Practice 8, Washington, D. C., 1971.

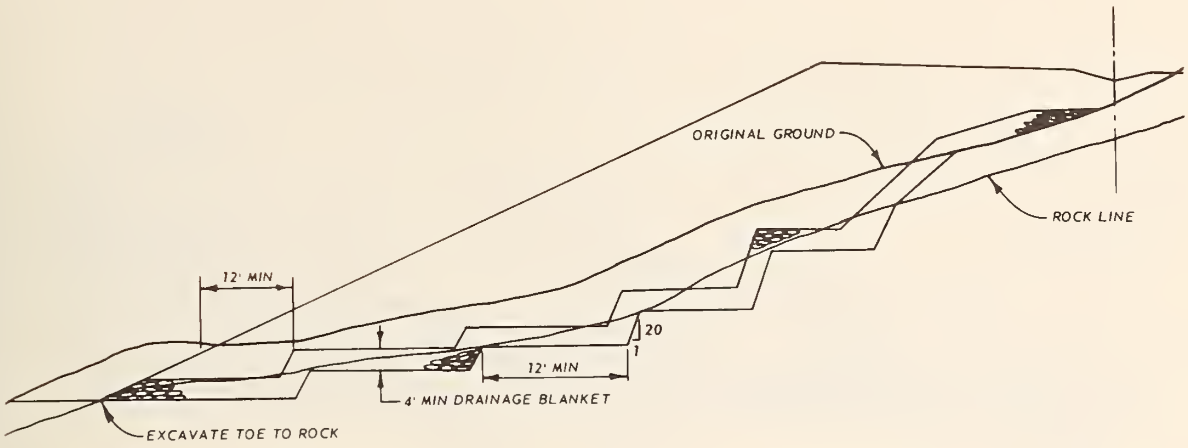


Figure 20. Bench design with rock drainage (courtesy West Virginia Department of Highways)

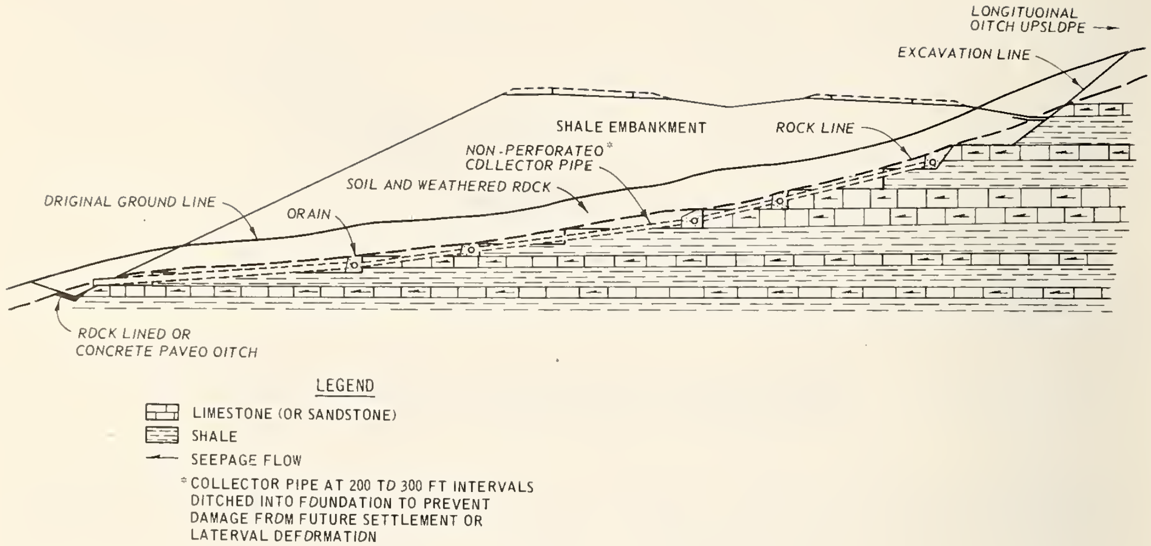
away from the embankment. The project engineer must still be relied on to require benching and/or drains at apparent seepage locations missed during the design investigations. Special drainage measures are needed for coal strata as illustrated in Figure 22. These locations should also be shown on the construction plans and provisions made for additional drains at locations missed during design investigations.

Material treatment and usage

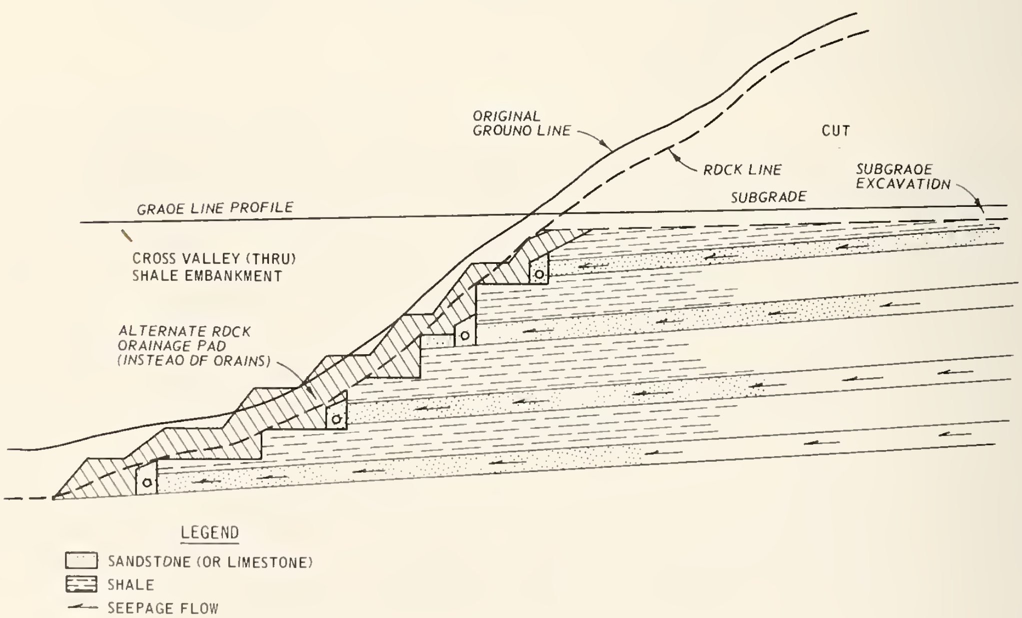
109. Material treatment and usage alternatives include breaking down all shale and harder rock for compaction as soil, selective excavation, or uncontrolled excavation. Wasting of shales can usually be avoided with proper treatment and usage. An exception might be extremely wet clay shales with high liquid limits that cannot be economically dried back by disking or other means. The above options, as discussed in Part II and outlined in Figure 2, depend on the amount of detailed information on formation stratification and shale durability, the economies of construction, and the degree of acceptable risk. A summary of alternatives corresponding to type of formation stratification is given in Table 10. Alternate thin layers of nondurable shale and limestone usually cannot be selectively excavated and need to be broken down and compacted as thin lifts. However, thick layers of rock and shales of different durability can be selectively excavated and placed in different portions of the embankment. Specifying the treatment and usage of shales is the key element in designing adequate shale embankments.

Embankment design

110. Important embankments. For major shale embankments (generally over 50 ft) especially on sidehill locations and other shale embankments with stringent requirements for stable slopes and little or no settlement, the following procedures should be followed:

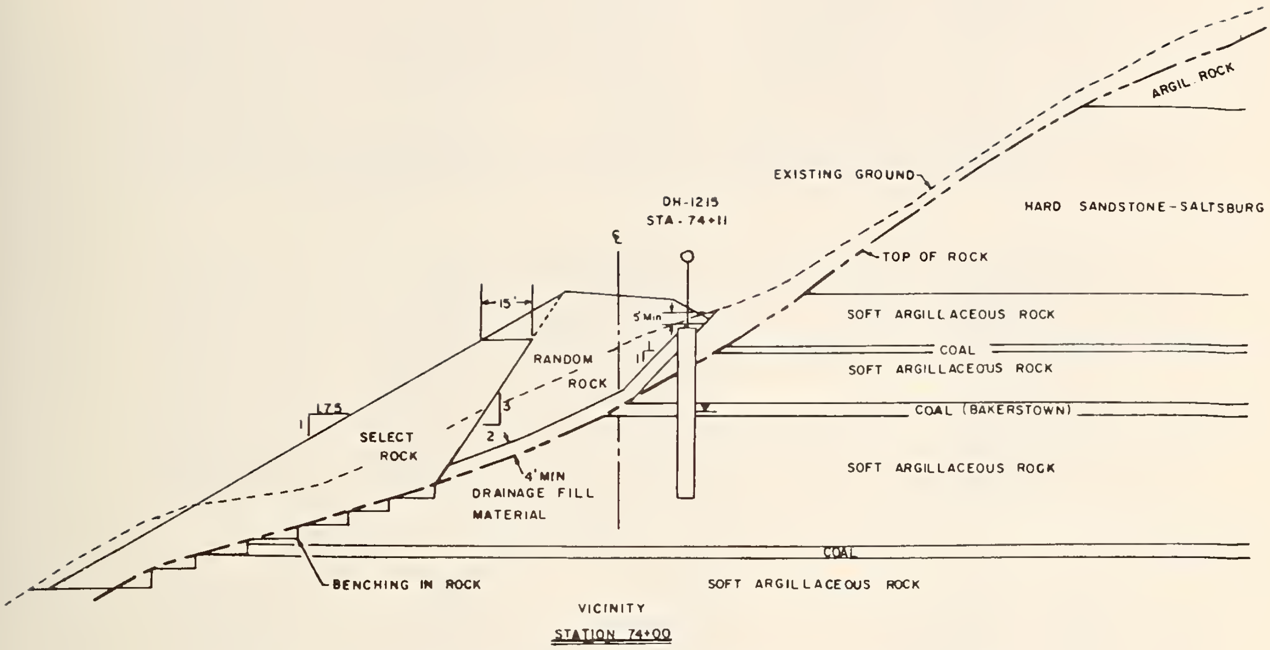


a. Longitudinal bench drainage

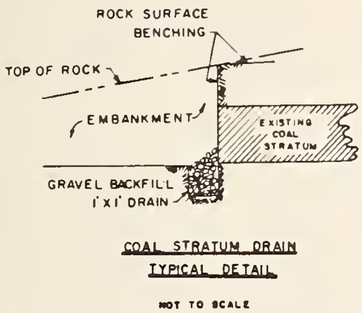


b. Transverse bench drainage, cut-to-fill

Figure 21. Bench drainage tailored to stratification of seepage layers



a. Sidehill embankment benching and drainage



b. Drainage measure for primary seepage stratum

c. Drainage of mined coal seam

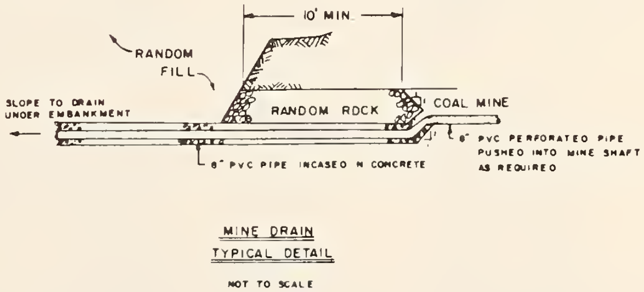


Figure 22. Examples of benching and special drainage measures

Table 10. Material Treatment and Usage Alternatives

	Stratification of Cut or Other Borrow Source	Shale Durability Classification	Excavation Treatment	Material Usage	Compaction Criteria
A.	Thin layers of shale and rock or steeply inclined shale and rock layers.	Nondurable (soillike, more than 40% of total cut volume).	Specify maximum excavated shale and rock size, or Specify maximum allowable shale and rock size for compaction and require breakdown on fill or raking to outer slope.	Place and compact as excavated on prepared foundation (e.g., required benching and drainage).	Specify maximum allowable loose lift thickness (8 to 12 in.*), moisture range, and relative compaction (percent maximum dry density), or Specify maximum allowable loose lift thickness (8 to 12 in.*), moisture conditions, compaction procedure, and type and minimum weight of compaction equipment, with provisions for test pads.
	Thin layers of shale and rock or steeply inclined shale and rock strata.	Durable (rocklike, 60% or more of total cut volume).	Specify maximum allowable excavated shale and rock size and maximum amount of minus No. 4 material.	Place and compact on benched sidehill foundations first to provide drainage. Require soil-like materials in upper part or central portion of embankment with rock in outer sections.	Specify maximum loose lift thickness (24 in.), type and minimum weight of compaction equipment, and compaction procedure, with provisions for test pads.
B.	Thick layers of shale or shale and rock that can be excavated separately.	Nondurable (soillike, more than 40% in a particular layer).	Specify maximum allowable shale and rock size and denote where particular layers are to be used in embankment.	Selective placement in specified locations in embankment with durable rock on foundation and outer sections of embankment.	Same as above for soillike shale. Durable rock per standard provisions or specifications.
		Durable (rocklike, 60% or more of particular layer).			Same as above for rocklike shale.
C.	Same as A or B above.	Classify as excavated (least desirable alternative).	Specify maximum allowable shale and rock size, or No special treatment; require end result compaction on all materials.	Selective placement (same as above). Place as excavated on prepared foundation (e.g. required benching and drainage).	Same as above for soillike or rocklike shales, or Specify lift thickness by maximum rock size and percent plus 6 in. and require 95% T-99 or CALDOT 90% relative compaction.

* Depending on type and weight of compaction equipment.

a. Classify different stratum within each cut or borrow source or entire cut sections according to excavation difficulty (paragraphs 96 to 98) and designate the use of each as either soilfill or rockfill, considering shale durability (Part IV).

b. Develop the best construction sequence scheme for using available quantities of excavation materials as soilfill and rockfill in different sections of the embankments.

c. Prepare embankment cross sections showing designated zones for use of excavated materials as soilfill and/or rockfill (Figures 11 and 22a), using slope inclinations recommended in Table 11.

d. Estimate probable excavated gradation (maximum size, percent plus 6 in. size, and percent plus No. 4 material) and determine the need for special gradation and/or excavation requirements (paragraph 112) for the lift thickness to be specified.

e. Assign laboratory compaction and shear strength tests needed on representative shales, using proportional gradation to model expected field gradations for soillike

Table 11. Recommended Slope Inclinations for Shale Embankments

Embankment Section	Compaction	Minimum Slope Inclination*
Soillike, homogeneous	Well compacted to at least 95 percent of AASHTO T-99 maximum density	2:1 (1V on 2H) 3:1 (1V on 3H) if good compaction cannot be assured
Soillike homogeneous: bridge approach embankments	Well compacted to at least 95 percent of AASHTO T-99 maximum density	3:1 (1V on 2H) to limit settlement at structures
Rock in outer sections to within 40 ft or less of grade	Well compacted (compaction procedure specified)	1.5:1 (1V on 1.5H)

* Flatter slopes may be required on weak foundations or where settlements due to lateral creep must be minimized. Stability analyses should be made to verify stability of important embankments especially on soil foundations or on colluvium or other residual soil slopes.

shale materials. Special procedural provisions should be considered for compaction of soillike shales with more than 35 percent gravel sizes (plus No. 4 sieve) and for durable shales used for rockfill. The above procedures should be initiated as soon as geologic profiles and sections in cuts or borrow areas can be prepared, following the field boring and sampling program. After material properties are determined (paragraphs 121 to 125), the safety factor should be determined (paragraphs 131 to 134) for typical sections and problem areas to verify stability.

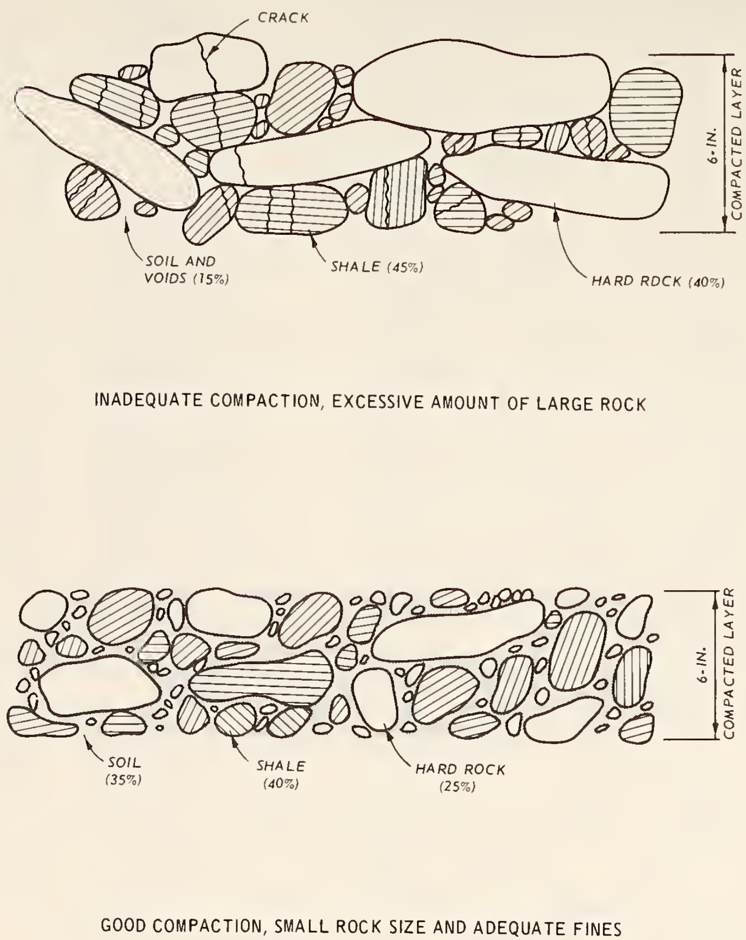
111. Routine embankments. Shale embankments in areas where a higher risk can be tolerated and where past practice in the area has proven satisfactory can be designed using normal procedures, provided good field supervision and inspection will be available during construction.

Gradation requirements

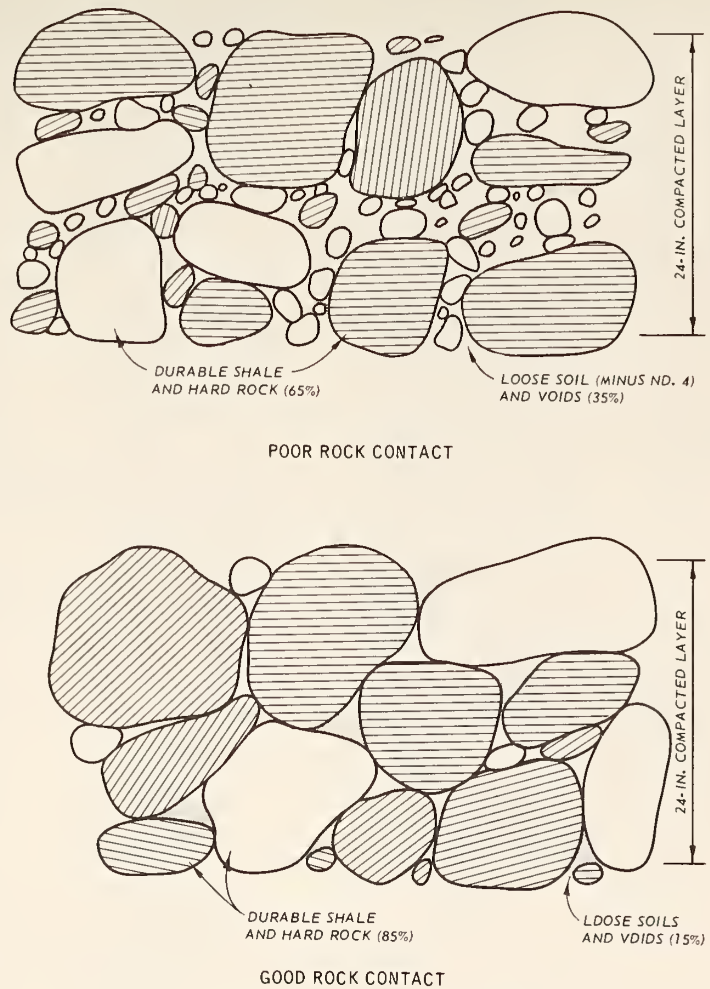
112. Gradation requirements for nondurable shales placed as soil should limit large rock sizes and provide adequate fines, while for durable shales placed as rockfill, excessive fines must be limited. For example, if a 10-ft-thick section of thin shale layers in a cut contained about 50 percent nondurable shale (which could soften and break down into a weak soil under infiltrating water after construction), the entire section should be considered soillike and compacted in thin lifts (8 to 12 in.). In this case, an excessive amount of large shale or hard rock sizes would prevent adequate compaction, as illustrated in Figure 23a. The excessive amount of large rock (upper drawing, Figure 23a) produces a loose, pervious structure. The shale pieces, cracked by stresses at contacts, would soften and break down further as water infiltrated down into the completed embankment. On the other hand, if the 10-ft-thick section contained 60 percent or more of durable shale, then the material could be used as rockfill. But in this case, an excessive amount of fine-grained material could prevent adequate contacts between durable rocks, as illustrated in Figure 23b. In the upper drawing, the loose soil between rocks would soften and deform under infiltrating water, resulting in large settlements, limitations on the percentage of large rock in nondurable shales compacted in thin lifts and the amount of fine-grained material in durable shales used in rockfill are discussed below.

113. Nondurable shales as soilfill. Compaction studies of minus 3-in. earth rock mixtures using an 18-in.-diameter mold by Donaghe and Townsend (1976)* showed that maximum dry density decreased significantly

* Donaghe, R. T. and Townsend, F. C., "Scalping and Replacement Effects on the Compaction Characteristics of Earth-Rock Mixtures," Soil Specimen Preparation for Laboratory Testing, STP 599, American Society for Testing Materials, Philadelphia, PA, Jun 1976.



a. Nondurable shale in soilfill



b. Durable shale in rockfill

Figure 23. Effects of rock size on nondurable shale and of soil (minus No. 4) on durable shale

when the gravel content exceeded 60 percent. As shown in Figure 24, the highest maximum dry density was 138 pcf for 40 percent gravel and 25 percent fines (minus No. 200 sieve), compared to 135 pcf for 60 percent gravel. When the amount of fines was reduced to 15 percent, the maximum dry density increased to 142 pcf.

114. Using the gradation curves from the earth rock mixture tests and assuming a maximum rock size of 12 in. for an 8-in. lift, a proportional gradation curve (paragraph 85) was constructed (using the offset distance A based on the difference between the maximum size, 12 in. to 3 in., as shown in Figure 24 for the dashed curve). This proportional curve indicates a well-graded material with 20 percent of plus 6-in. size rock.

115. Since a 6-in. size is easily recognized in the field and rock larger than 12 in. should be prohibited or limited to a smaller percentage, criteria limiting plus 6-in. rock size to less than 20 percent should be used as minimum. Hard nondurable shales that do not contain sufficient fines may require an additional limitation of about 60 percent on the plus 1-in. size (or a requirement for about 40 percent, minus 1-in. material).

116. The use of heavy compaction equipment on the rocky mixture of nondurable shale and hard rock shown in Figure 23a (upper drawing) would not produce adequate density because hard rock, such as limestone, would not break down. Conversely, small rock sizes and soil can be well compacted in thin lifts using conventional compaction equipment.

117. Durable shale as rockfill. Durable shales and rock used as rockfill require good contact to achieve a stable mass that will not deform or settle. As illustrated in the upper drawing in Figure 23b, the larger shale and rock is "floating" in loose soil. It would be practically impossible to obtain good compaction of the soil even with very heavy equipment. Thus the loose soil structure would compress and deform with time under infiltrating water and result in large settlements. A stable of mass of large rock is attained, as illustrated in Figure 23b (lower drawing), when large rock pieces are pushed together to form a large number of contacts. To achieve the desired clean rock, the amount of soil (minus No. 4) should be limited to not more than 20 percent for lifts as thick as 24 in.

Excavation requirements

118. One means of obtaining gradation requirements would be to specify excavation procedures, such as type and extent of ripping or blasting as discussed in Part V, with provisions for test excavations and test pads at the start of construction. Alternatives include the following:

a. Specifying, for each cut or different strata within a cut,

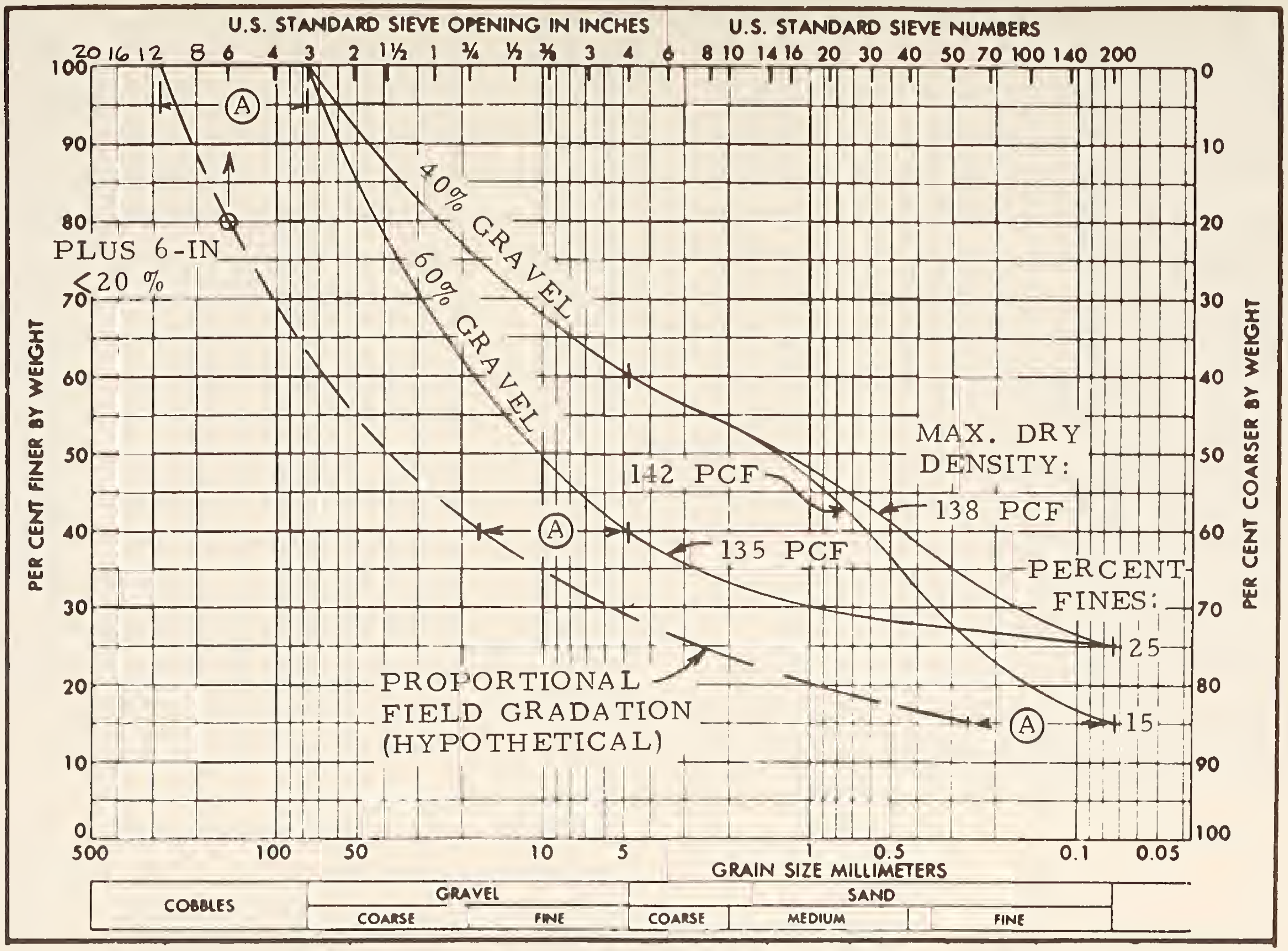


Figure 24. Basis of gradation requirements for nondurable shales used as soilfill

the maximum allowable rock size and percent plus 6-in. rock for nondurable shale and maximum percent of minus No. 4 for durable shale used as rockfill.

b. Specifying that oversize, nondurable shales be broken down on the fill by one or more of the following: (1) watering and disking, (2) using rollers with small area (6 in.^{2}) or pointed feet (shale breakers), (3) using hammer equipment, such as a drop hammer mounted on the front of a rough terrain fork lift body, and that oversize hard rock be raked to the outer rockfill section or the outer slope of the embankment. The main objectives are to (1) ensure that nondurable shales are well compacted into dense, relatively impervious layers that will prevent infiltration by surface water or subsurface seepage and (2) ensure numerous contacts between individual pieces of durable shales and harder rock used in rockfill by excluding an excessive amount of soil.

Compaction requirements

119. The wide variation of in-place densities of coarse-graded shales, even in carefully controlled shale test pads, indicates that end result specifications can be completely misleading for shale compacted as soil. An exception would be for softer shales that break down into fine-grained soil. The use of procedural specifications or procedural special provisions for shale embankments can reduce field testing requirements once the procedure is developed. Shale test pads included as part of construction are the best means of establishing adequate compaction methods and procedures. Recommended procedures for shale test pads are given in Appendix A.

120. Carefully controlled shale test pads in Indiana described by Sisiliano et al. (1978),* have verified the need for procedural type special provisions given in Figure 25, which are in current use. An important operation is adding water to slaking shales found dry and the use of heavy duty disks to mix in water and break down shale pieces. Two types of heavy compaction equipment have generally been required to obtain adequate compaction (95 to 100 percent of T-99). Three coverages with a 30-ton static tamping-foot roller, followed by two coverages of a vibratory tamping-foot roller with a minimum compactive effort of 55,000 lb are used in Indiana. Following Ohio's procedure, the Corps of Engineers, on several projects, has used three coverages with a 30-ton static tamping-foot roller, followed by two coverages with the

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* Sisiliano, W. J. et al., "Report of a Shale Test Pad, R-Contract No. 10783, TQF-Project No. 105-1(1) Const., S.R. 145 in Orange County," Indiana State Highway Commission Division of Materials and Tests, Soils Department, Indianapolis, IN, May 1978.

Shale Compaction Special Provisions

When shale, shale and weathered rock mixtures, or weathered rock are encountered that is to be used in the embankment, the compaction shall be accomplished with an approved vibratory tamping-foot roller in conjunction with a static tamping-foot roller. The minimum weight for the static tamping-foot roller shall be 60,000 pounds. The minimum compactive effort for the vibratory tamping-foot roller shall be 55,000 pounds defined as the rated centrifugal force at operating frequency plus the static drum weight.

Unless otherwise approved in writing, the embankment lifts shall receive a minimum of three (3) coverages with the static roller and two (2) coverages with the vibratory roller. Maximum speed of rollers shall not exceed 5 mph. This compaction procedure shall, unless directed otherwise, be checked by constructing test pads on the materials involved on this project, and the number of passes adjusted if necessary to obtain the desired density with the minimum number of passes of each roller. No additional compensation will be allowed for the compaction equipment and requirements specified herein, the costs of which shall be included in the various items of the contract.

Under this procedure the shale and weathered rock mixture, or weathered rock will be placed in eight- (8-) inch maximum embankment lifts, and unsoft rock will be placed in two- (2-) foot embankment lifts, unless otherwise approved. When shale with high slaking characteristics is found dry, the Contractor will be required to apply water to this material in order to facilitate its compaction. Water shall be added by spray bar attachment exclusively. The amount of water will be that required to achieve approximately optimum moisture for the particular material involved. The added water shall be uniformly mixed with the material for the entire depth of the lift by disking.

Excavation and blasting methods shall be such that the above described materials that are to be placed in eight- (8-) inch lifts will be excavated selectively, and placed in the embankment selectively; and not intermixed with rock, weathered rock, or other rocklike material that would preclude placing and compacting the material in eight- (8-) inch lifts. Blasting of the different strata simultaneously will not be permitted, unless otherwise authorized in writing.

Water required and furnished by the Contractor to facilitate the compaction of the soft shale will be paid for at the unit price bid per thousand gallons (M) for "Water for Shale" which price shall include full payment for furnishing and applying the water, mixing, and labor and equipment.

Figure 25. Example of shale compaction special provisions (based on provisions used in Indiana, courtesy of Indiana State Highway Commission)

50-ton rubber-tired (4-wheel) roller. The above weights are minimum requirements for 8-in. loose lifts. The type (such as square or pointed) and size (such as 6 in.^{2}, maximum) of tamping feet for static rollers may need to be specified, to help break down rocky shales.

Determination of Shale Material Properties

121. Properties needed for design of embankments with soillike shales are compaction, compressibility, permeability, and shear strength. The expected gradation of fill materials should control the determination of these properties. For soft shales that break down into fine-grained soil, normal soil testing procedures are adequate. Harder shales that remain coarse-graded after excavation can be tested, using proportional gradation (Figure 13) to prepare representative small-scale specimens. The use of index tests, such as slake durability to predict settlement of well compacted soillike shales on the basis of soaked compression tests (paragraph 88 and Figure 15), appears promising. For embankments as high as 200-ft, large-scale tests and monitoring may be desirable as described by Smith and Klieman (1971)* and Chang and Forsyth (1973),** or as described in paragraph 129.

Compaction properties

122. Compaction properties (maximum density and optimum water content) should be based on AASHTO T-99, Method D for coarse-graded shales. New material should be used for each test point. Proportional gradation (Figure 13) or scalping of plus 3/4 in. (with no replacement) can be used for shales with oversize particles (Vol. 4). Water contents for compaction testing should range from air-dry to near saturation and include a sample near the in situ water content. This range is important in establishing whether a shale can be well compacted at its natural water content or whether mixing in of additional water is needed to achieve adequate field compaction.

Compressibility and permeability

123. The compressibility of well compacted shales at water contents of 6 to 12 percent is small. However, compression after soaking can be large and can result in excessive settlement with time. Compression index tests should be performed on compacted samples, as described

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* Smith, T. and Klieman, W. K., "Behavior of High Embankment on US-101," Highway Research Record No. 345, Transportation Research Board, Washington, D. C., 1971.

** Chang, J. C. and Forsyth, R. A., "Stresses and Deformations in Jail Gulch Embankment," Highway Research Record No. 457, Transportation Research Board, Washington, D. C., 1973.

in Vol. 4, to estimate possible settlements and the need for greater compaction. Correlation of soaked compression with slake-durability index (Figure 15) could lead to considerable reduction in laboratory testing for routine embankments. Checking the permeability of compacted shales is also important. If the compacted shale has a low permeability (e.g., 4×10^{-8} cm/sec), it will probably resist migration of infiltrating water.

Shear strength

124. The shear strength of soillike shales can be determined from triaxial compression or direct shear tests on samples molded to expected field densities and water contents. Proportional gradation (Figure 13) or scalping of plus 3/4-in. sizes can be used to model coarse-graded shales. For a maximum particle size of 3/4 in., the minimum sample size for triaxial compression tests is about 3 in. in diameter by 6 in. high and 3 by 3 in. by 3/4 in. for direct shear testing. Strength and deformation characteristics of a hard nondurable Indiana shale (New Providence formation), compacted to various degrees, has been studied by Abeyesekera (1977).*

125. Shear strengths of shales in older embankments and well compacted shales, as shown in Figure 26, will usually be adequate for embankment stability,** with slopes of 2:1 or flatter. However, as illustrated in Figure 27, larger strains may occur at a smaller compaction effort. Considerably more deformation (Figure 27) and thus settlement could occur in material compacted to 93 percent of T-99 density before adequate shear strength would develop. Since the main problem is to limit settlements, the solution may be to require greater compaction effort in the field or special measures to prevent wetting by infiltrating water (paragraphs 136 and 137).

Selection of Design Values

126. The degree of conservatism used in selecting design values for shale embankments depends on a number of factors. These factors include:

a. Amount of past experience with similar shales.

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* Abeyesekera, R. A., "Stress-Deformation and Strength Characteristics of a Compacted Shale," Joint Highway Research Project JHRP-77-24, Purdue University, West Lafayette, IN, Dec 1977.

** Major slides have generally resulted from overstressing of weak clays or shales at or below the base of the embankment on sidehill locations.

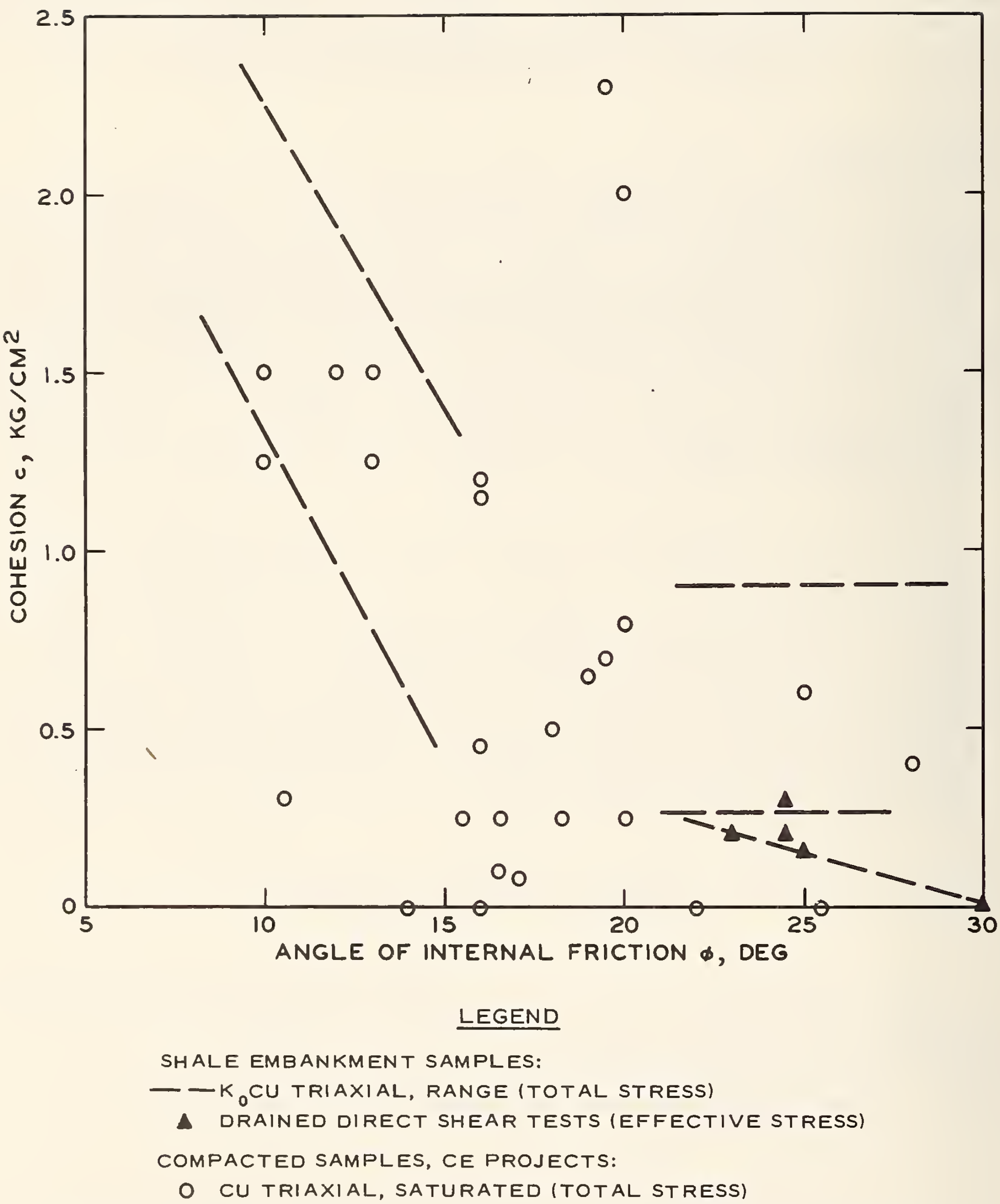
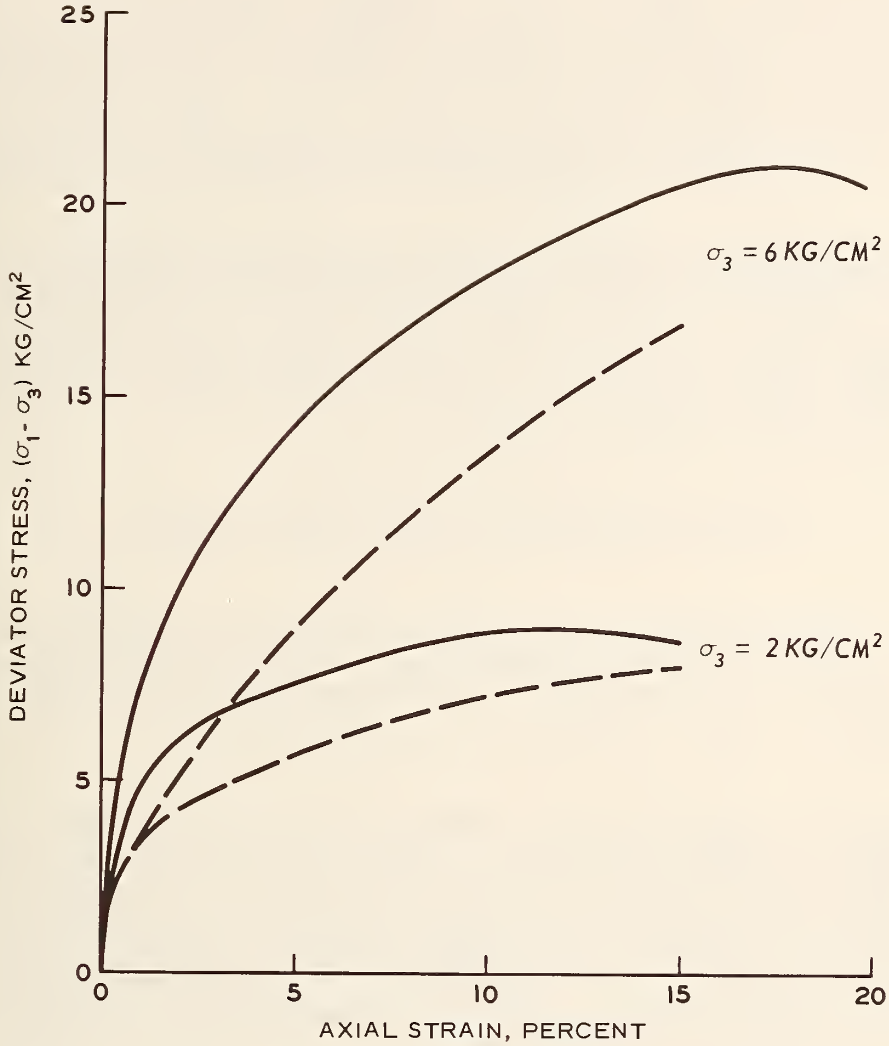


Figure 26. Range of strength parameters (c, ϕ) for compacted shales based on maximum deviator stress



LEGEND

——— COMPACTED TO 100% T-99 (124 PCF)

— — — COMPACTED TO 93% T-99 (116 PCF)

NOTE: MATERIAL TESTED WAS A REDDISH SILTSTONE FROM RAYSTOWN DAM, PA. TESTS PERFORMED ON BACK PRESSURE SATURATED SAMPLES, 14-IN. IN DIAMETER BY 28-IN.-HIGH, OF MINUS 3-IN.-SIZE MATERIAL.

Figure 27. Example of increase in strain with decrease in compaction effort (CU triaxial compression, total stress)

b. Importance of project and degree of allowable risk.

c. Uniformity of shale formations along project route.

d. Amount and representative nature of tests performed.

e. Amount of inspection and control to be exercised over excavation, placement, and compaction during construction.

Complete information developed during design studies would allow a minimum of conservatism. However, if adequate control during construction was not possible, a conservative approach including increased drainage measures, greater compaction effort, and flatter slopes may be necessary. For example, where settlements must be minimized, such as at bridge approach embankments, compaction equivalent to 95 percent of T-99 and 2.5:1 or 3:1 slopes may be required.

Design Evaluation Criteria and Techniques

127. Design analyses applicable to shale embankments include settlement, slope stability, and stability improvement techniques (special design features, paragraph 105). These items are discussed below.

Settlement

128. Shale embankment settlement is the most common problem but the hardest to predict and control. Nonuniform settlements reflect the heterogeneous nature of shale embankments and erratic paths followed by infiltrating water. For normal design standards, gross estimates of settlement can be obtained from soaked compression tests on compacted samples (paragraph 88) or on the basis of the slake-durability index (Figure 15) when sufficient correlation data have been developed. Large estimated settlements (greater than 6 in.) would indicate the need for increased compaction to reduce settlement.

129. Special embankments such as those adjacent to or crossing reservoirs or lakes may warrant more detailed studies. If shale materials will be relatively uniform within the embankments and can be represented by modeled gradations, then tests and analytical procedures described by Nobari and Duncan (1972)* can be used to determine stresses and deformation due to wetting in the assessment of long-term performance.

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* Nobari, E. S. and Duncan, J. M., "Effect of Reservoir Filling on Stresses and Movements in Earth and Rockfill Dams," Contract Report No. S-72-2, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS, Jan 1972.

Slope stability

130. Slope stability usually is not critical within shale embankments, except for shallow sloughing caused by seepage or wet slope areas and sidehill foundation failures. However, deformations required to mobilize adequate shear strength for equilibrium can cause large settlements (paragraph 125). Thus, analyses of stress-strain properties and deformation in connection with settlement as described above may be more important than slope stability against slides.

131. The range in factor of safety (FS) for various combinations of shear strength parameters (c, ϕ) using a complete equilibrium method (Morgenstern and Price) is illustrated in Figure 28. Plots for different embankment heights with a constant 2:1 slope include a shaded area representing the shear strength range for shale embankment samples tested during this study (Vol. 4). These plots can be used to quickly relate range in factor of safety with the range in shear strengths selected from tests or estimated from Atterberg limits, as described by Hopkins et al. (1975)* or Townsend (paragraph 81). Hopkins also discusses effects of water and geometry changes on slope stability.

132. The effect of a water surface located at the embankment midheight is shown in the plots on the right in Figure 28. For a 100-ft-high, 2:1 slope, the minimum strengths (ϕ = 15- to 20-deg range) of samples from older shale embankments (15 years old) correspond to an FS of 1.2. For a 200-ft-high slope, this minimum strength indicates an FS at or below 1.0. Flatter slopes would obviously have a higher FS.

133. The effect of slope height on required shear strengths for a given FS of 1.2 is shown in Figure 29. For shale embankment heights less than 100 ft, the long-term FS on stability against embankment slides would exceed 1.2, considering the lower limit of strengths for aged shale embankments. Shale embankments with heights greater than 100 ft should be checked since the factor of safety would be near 1.0 for strengths of 15 to 20 deg with cohesion values of 0.5 to 0.3 kg/cm^2, respectively.

134. Stability charts based on the simplified Bishop Method, such as those by Huang (1975, 1978),** can be used for homogeneous slopes.

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* Hopkins, T. C., Allen, D. L., and Dean, R. C., "Effects of Water on Slope Stability," Research Report 435, Division of Research, Bureau of Highways, Department of Transportation, Lexington, KY, 1975.

** Huang, Y. H., "Stability Charts for Earth Embankments," Transportation Research Record 548, Transportation Research Board, Washington, D. C., 1975.

Huang, Y. H., "Stability Charts for Sidehill Fills," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 104, No. GT5, May 1978.

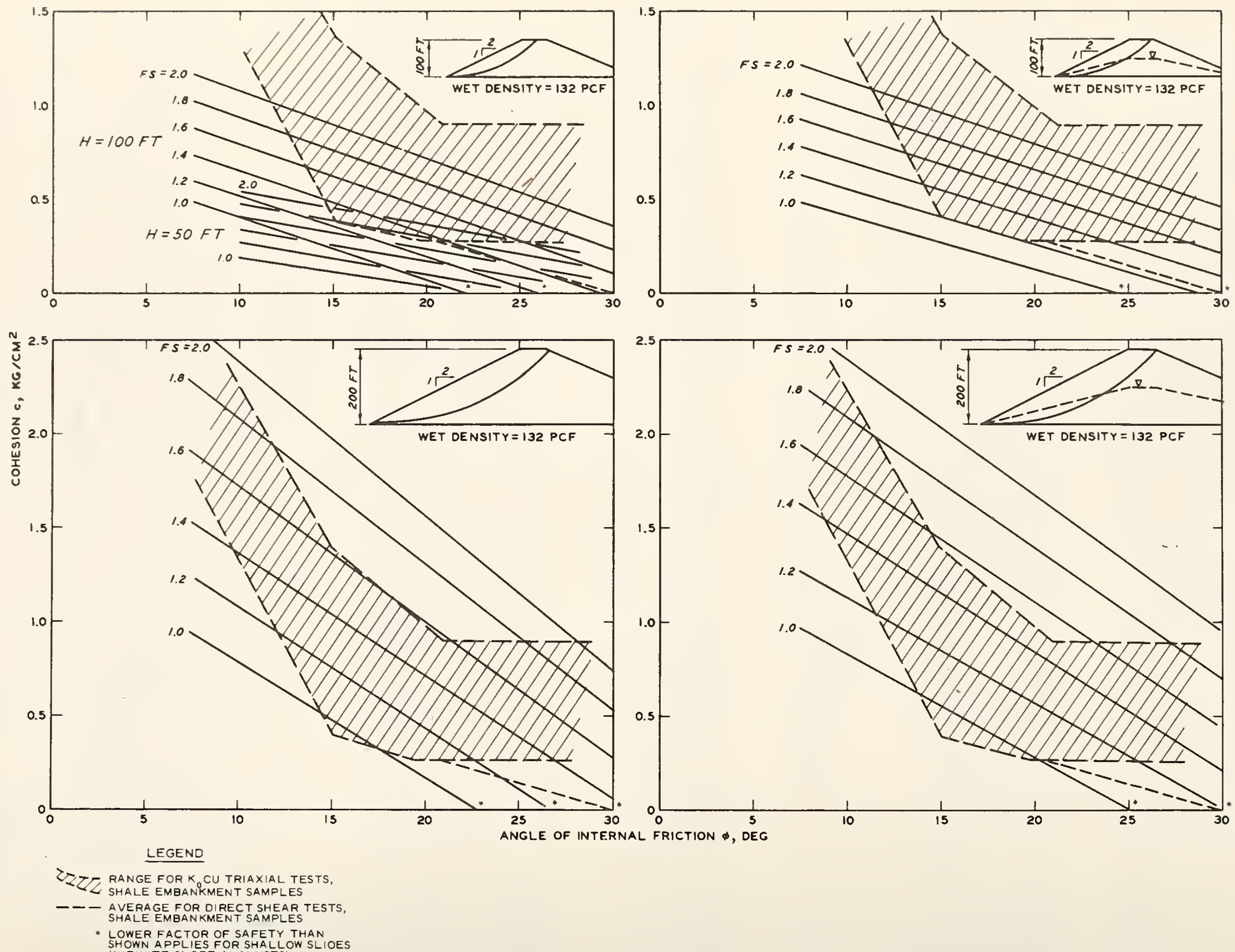

Figure 28. Shear strengths related to factor of safety for shale embankments with 2:1 side slopes

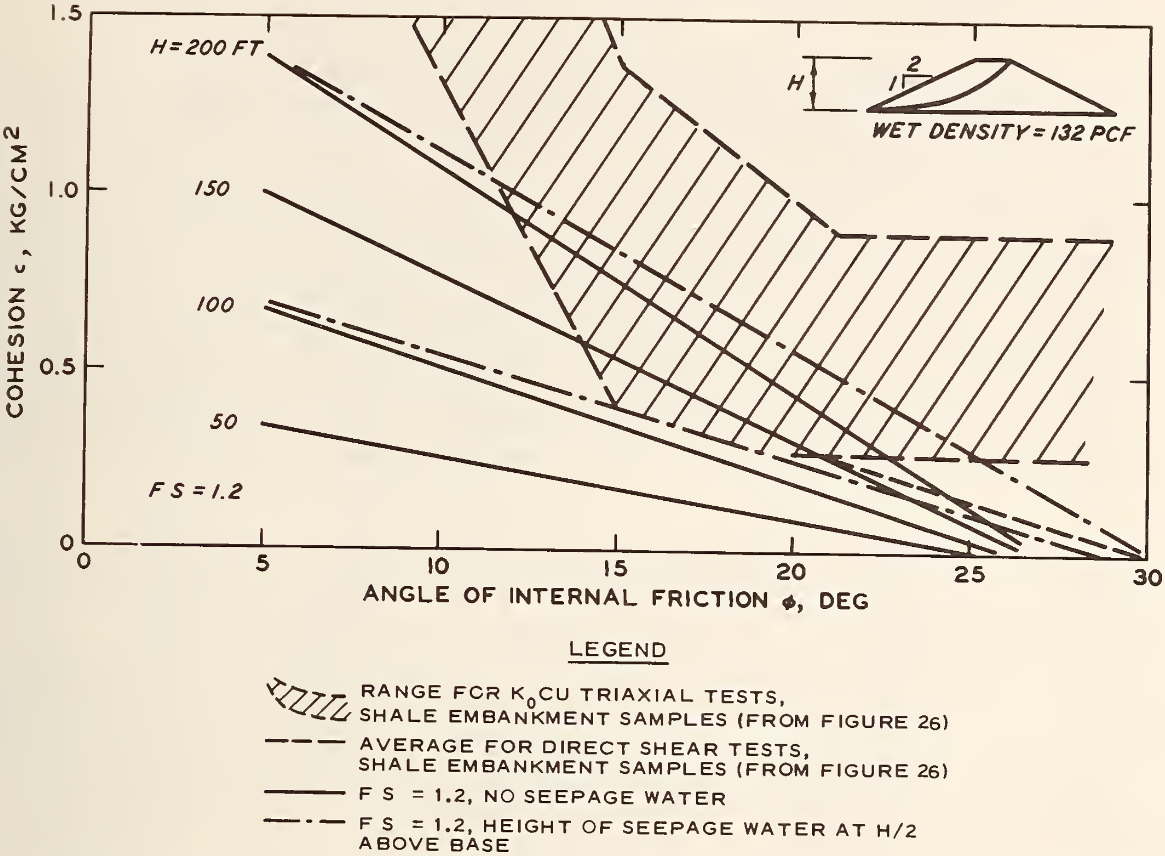

Figure 29. Shear strengths related to slope height for FS = 1.2 and 2:1 side slopes

It should be realized that values of the pore-pressure ratio, r_u, greater than zero imply a phreatic surface at a proportional height above the entire slip surface, depending on the value of r_u and ratio of total unit weight of soil to unit weight of water. This definition may not represent the actual water surface location. For nonhomogeneous slopes, Bishop's simplified method of slices computerized by Yoder and Hopkins (1973)* can be used. This program requires a phreatic surface to define pore pressures and is more realistic than using r_u.

Stability improvement methods

135. Special design features to improve the stability of shale embankments (paragraph 105) are discussed in Part IX in connection with remedial treatment. Only brief comments on design evaluation criteria

* Yoder, S. M. and Hopkins, T. C., "Slope Stability Analysis: A Computerized Solution of Bishop's Simplified Method of Slices," Research Report 358, Division of Research, Department of Transportation, Lexington, KY, Feb 1973.

and techniques important to shale embankments are given here.

136. Underdrains, spring drains, and drainage layers. Underdrains, spring drains, and rock drainage layers used to drain excess seepage from the embankment foundation must meet two criteria. These are sufficient permeability to carry away the water and a grading that will prevent clogging by fines entering from the foundation or the overlaying embankment. Of particular importance is proper grading to prevent movement of fine-grained shale and soil out of the embankment and into the drain, causing voids in the embankment and clogging in the drain. Proper grading can be accomplished by (a) "choking" the top of drainage layers by breaking down rock or aggregate into smaller pieces, (b) using filter fabric as a protective but permeable screen between coarse and fine materials,* and (c) using different gradations of aggregate from coarser to finer, following standard criteria (see Cedergren, 1977** for general guidance).

137. Surface drainage measures. Paved median ditches, impermeable barriers to prevent infiltration through shoulders and wide medians, and pavement subdrains are primary measures for reducing settlements in shale embankments. These measures should be used when adequate compaction of coarse-graded shales cannot be ensured. Even if select material is well compacted in the top several feet, forming a relatively impermeable cap, differential settlement within the shale embankment can cause vertical cracks in the cap that will feed surface water downward. An example of recommended barriers as a remedial measure in Indiana is shown in Figure 30.

138. Berms and shear trenches. Berms to provide added stability (as a counterweight) and shear trenches to increase shear resistance of weak foundation materials require adequate drainage and compaction. Examples are shown in Part IX.

139. Retaining structures. The main requirement of all retaining structures is adequate drainage to prevent buildup of hydrostatic pressure behind and beneath the structure. Rock buttresses can deform and fail when voids are eroded behind the rock by seepage out of the slope or by surface water running down the shale or soil backslope of the rock. A graded filter or filter fabric should protect the foundation backslope of the buttress. Filters are required behind crib walls and gabion walls to prevent erosion of fine-grained materials into drainage paths in the rock compacted into the wall, thus blocking the drainage paths. Gabion walls can accommodate settlement deformations if properly

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* Federal Highway Administration, "Sample Specifications for Engineering Fabrics," Report No. FHWA-TS-78-211, Implementation Division (HDV-22), Offices of Research and Development, Washington, D. C., 1978.

** Cedergren, H. R., Seepage, Drainage and Flow Nets, 2d ed., John Wiley, New York, 1977.

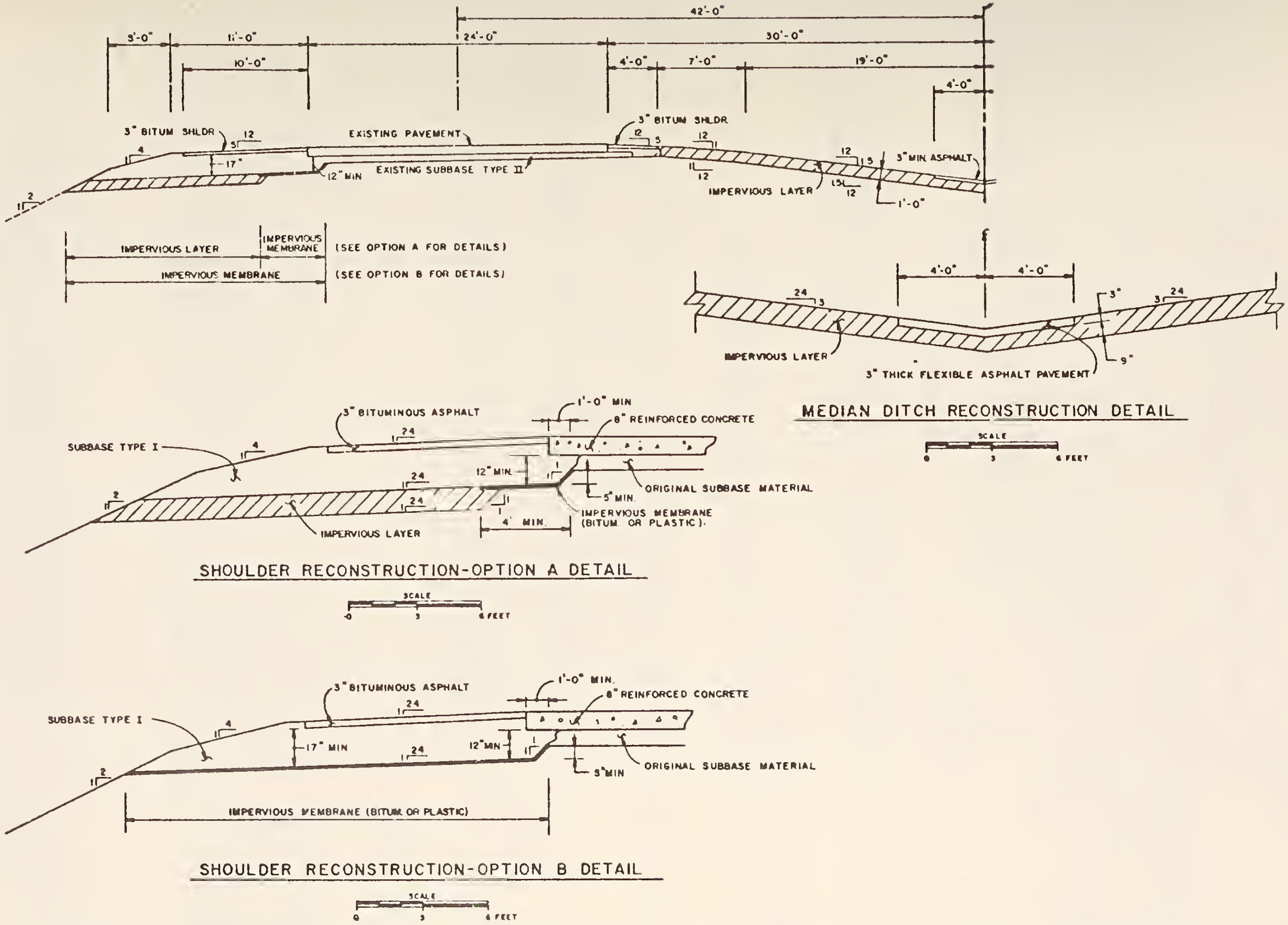

Figure 30. Example of impervious median and shoulder measures to prevent infiltration (courtesy Indiana State Highway Commission)

designed against failure (Schuster, 1974).*

Construction Provisions and Control Techniques

Construction provisions

140. Construction drawings and special provisions should contain specific information on the following items.

- a. Foundation benching and drainage measures: location and type of underdrains, spring drains, and drainage layers (blanket drains) as described in paragraph 107 of this section.

- b. Designated zones or strata in cuts for use as soilfill or as rockfill (Figure 11), depending on durability classification and specified gradation criteria (paragraphs 112 to 117) or excavation methods to obtain desired gradations.

- c. Location of soilfill and rockfill sections within embankments by stations and typical cross sections.

- d. Placement criteria, lift thickness, and treatment of soil-like shale before compaction to include breaking down or raking large rock to the outer edge, adding water by spraying, and disking with heavy duty disks to mix in water and help break down larger shale pieces (Figure 25).

- e. Compaction procedures including types and minimum weights of equipment, number of passes, and maximum speed (Figure 25).

Control techniques

141. Control of shale embankment construction by enforcement of special provisions, specifications, and contract drawings is essential. All the best testing and design work is lost if good construction control is not available. A primary step in achieving good construction control of shale embankments is to provide the project engineer's staff, particularly grading inspectors, with written guidance or instructions on important aspects of the project and critical areas for inspection.

142. General information is included in construction manuals used

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* Schuster, R. C., "Gabions in Highway Construction," Special Report 148, Transportation Research Board, Washington, D. C., 1974.

in many states. However, special instructions are necessary for each project because of varying shale conditions. Descriptions or expedient tests and procedures for identifying shale strata to be treated as soil should be included. Information should be given on how much breakdown of shale and rock is expected during excavation, approved procedures for cross-hauling, placement, treatment prior to compaction, compaction equipment types and methods, and check tests or measurements to verify adequacy of compaction. Visits by a geotechnical staff member at least once a week and visual observations and photographs from a helicopter can give a quick assessment of existing conditions. Specific control techniques for various phases of shale embankments are given in the next section (Part VII) on construction.

Instrumentation and Observation Requirements

143. Monitoring of critical shale embankments should be considered for two reasons:

a. The early detection of settlement and/or stability problems and application of remedial measures before a complete failure occurs.

b. To verify predicted performance and build a working knowledge of the most economical methods for construction of adequate shale embankments.

Shale embankments over 50 ft high with a low margin of safety should at least have settlement markers along the shoulders of the roadway (e.g., just inside the guard rail line). The markers should be surveyed two to three times per year (especially following periods of heavy rainfall or wet seasons) to detect increases in rate of settlement. Settlement increases could warrant a further investigation to define the cause of distress (Part VIII) and potential hazard. Depending on local experience, observations may need to be continued for 6 to 10 years or at increasing time intervals until additional settlement is insignificant.

144. A comprehensive reference on types of instruments and installation procedures is EM 1110-2-1908.* Comprehensive information on instrumentation with particular emphasis on accuracy of measurements is contained in Highway Focus, Vol. 4, No. 2, June 1972. A brief summary of basic instrumentation is given in Synthesis of Highway Practice 8, "Construction of Embankments," Highway Research Board, 1971.

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* U. S. Army Corps of Engineers, Instrumentation of Earth and Rockfill Dams, Part 1 of 2, "Groundwater and Pore Pressure Observations," Aug 1972, and Part 2 of 2, "Earth-Movement and Pressure Measurement Devices," Nov 1976, Engineer Manual, EM 1110-2-1908, Washington, D. C.

Surface movements

145. Surface movement markers, such as spikes along the roadway and concrete or iron pipe markers on the side slopes, provide valuable information. Comparative grade-line profiles and cross-section profiles with horizontal control surveys can be used to determine whether settlement only is occurring (all movement downward) or if slope bulging is involved (slope markers moving outward). Comparative aerial photography can be used as a rapid means to survey roadway and embankment slope movements.*

Subsurface movements

146. Lateral movements in shale embankments can be monitored using slope inclinometers. Settlement versus depth can also be monitored when sleeve-jointed casing and settlement probe are used. A pipe (2 to 4 ft long) placed in the bottom of a cased boring (or slope inclinometer boring) and attached to the surface by a flexible cable can be used to measure the depth to the deepest developing slide zone (bend in the casing restricts further upward movement of pipe). Slope inclinometer casing placed in shale embankments should be grouted in place, since sand/gravel backfill might not stabilize for several weeks or months. Slope inclinometer casing should also be surveyed with a spiral meter to correct movement data for spiraling of grooves.

Seepage

147. Cased borings used to measure water levels can only indicate a free-water surface. Casing that has penetrated a drainage layer at the base of the embankment could be used to monitor the continued operation of the drainage layer (water surface rising several inches above bottom of casing). However, detection of seepage higher up in the embankment or a perched water zone would require the installation of piezometers at different depths (usually at three depths, 1/4, 1/2, and 3/4 of embankment height). Rapid monitoring of seepage from shale embankment slopes can be accomplished by periodic remote sensing (paragraph 30).

Special observations

148. Direct reading nuclear-density probes that can be lowered to depths of 50 to 100 ft are available. These devices can be used to measure in-place water contents and densities and changes with time as described in Vol. 4. However, unless the probe is omnidirectional,

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* Roth, L. H., Cesare, J. A., and Allison, G. S., "Rapid Monitoring of Coal Refuse Embankments," (prepared by CH2M Hill) Final Report, Contract No. H0262009, U. S. Bureau of Mines, Spokane Mining Research Center, Washington, D. C., Jun 1977.

consistent data may be difficult to obtain. (Directional probes with a spiraling cable would cover different areas of the boring at a given depth on successive surveys.) Locations of wet zones and decreases of in-place densities could help in locating weak zones and seepage patterns. This information could be used in planning remedial drainage measures (by defining the probable seepage source from the transverse foundation slope near a cut or from infiltrating surface water).

PART VII: SHALE EMBANKMENT CONSTRUCTION

Construction Grading Sequence

149. Construction of shale embankments must be tailored to meet the main requirements, as discussed in Part II, of (a) breaking down and compacting nondurable shales in thin lifts to produce dense, relatively impermeable layers and (b) providing good drainage at the base of the embankment to prevent harmful seepage saturation or buildup of hydrostatic water pressures. These requirements dictate increased use of selective excavation and placement as directed by the plans and special provisions. Consequently, the overall grading sequence becomes a major consideration. Surface soils and weathered shales removed from sidehill benches and the top of cuts need to be compacted in cross valley embankments where foundation drainage layers are not required, or in the upper portion of embankments nearing completion. Stockpiling may be necessary at the start of construction. Hard rock and hard durable shales need to be used as drainage layers on sidehill benches and on transverse foundation slopes. These requirements cannot be met by the usual construction method of excavating material in a cut from the top down and placing it in the next fill from the bottom up.

150. The project engineer and his staff and the contractor should have a thorough understanding of the grading sequence scheme. When the plans include geotechnical profiles and sections identifying usage of particular strata from cuts in the embankment as shown by typical sections, preconstruction training of the project engineer's staff should cover type and extent of selective excavation and placement in addition to the following:

- a. Shale identification and durability classification (expedient field tests such as jar-slake tests, paragraph 67 and Figure 12).
- b. Required benching and installation of bench drains, subdrains, and springdrains.
- c. Influence of geology on required excavation and excavation methods needed to achieve required gradation for adequate compaction.
- d. Planned use of field test pads.
- e. Special provisions for excavation, placement, and compaction of soillike shales, durable shales, and durable rock.
- f. Compaction methods and equipment requirements.

g. Compaction control procedures and criteria.

h. Special construction features for gabion, reinforced earth, and crib walls, rock buttresses, berms, or shear trenches, and special drainage measures.

Frequent coordination is required between the geotechnical, design, and construction staffs during construction to insure adequate drainage and good compaction, to prevent undesirable mixing of nondurable shale or soil with rock, and to prevent hard, nondurable shale from being used as rockfill. Coordination is also required to solve unforeseen problems, such as less than anticipated quantities of durable rock needed for drainage layers, requirements for additional test pads to resolve compaction problems, or difficulties in classifying the durability of shale strata.

Foundation Preparation

151. An important part of shale embankment construction is "keying" the embankment into sloping ground surfaces by benches and installation of bench drains, underdrains, and springdrains, to intercept all potential subsurface drainage entering the foundation area.

Benching

152. The plans should be carefully followed in preparing benches into unweathered (Table 6) shale or rock (Figures 20 to 22). Excavation of benches starting at the lowest elevation and progressing up slope will allow orderly progression of embankment construction in horizontal layers from the bottom up. This procedure, illustrated in Figure 31, requires incremental installation of the drainage rock layer or bench drains. The alternative is to excavate all benches and install the drainage layer or drains before placing and compacting shale fill. An inspection aid for adequate bench depth is visual observation of unweathered rock in the uphill side of each bench.

Drainage layers

153. Drainage layers (or rock drainage pads) of sound, durable rock [2 to 4 ft] thick must be free-draining. Thus, the amount of soil and fines should be limited to less than 10 to 15 percent. Shale and siltstone should not be used for drainage layers unless their durability can be verified by a geologist or by tests during construction (Figure 12). The top surface of drainage layers must also be graded to provide a filter zone. This zone is needed to prevent migration of fines in overlaying compacted shales or soils into the rock, thus clogging the drain. One means of achieving a filter zone is by breaking down the surface rock under dozer treads to form a "choke" layer. An alternative

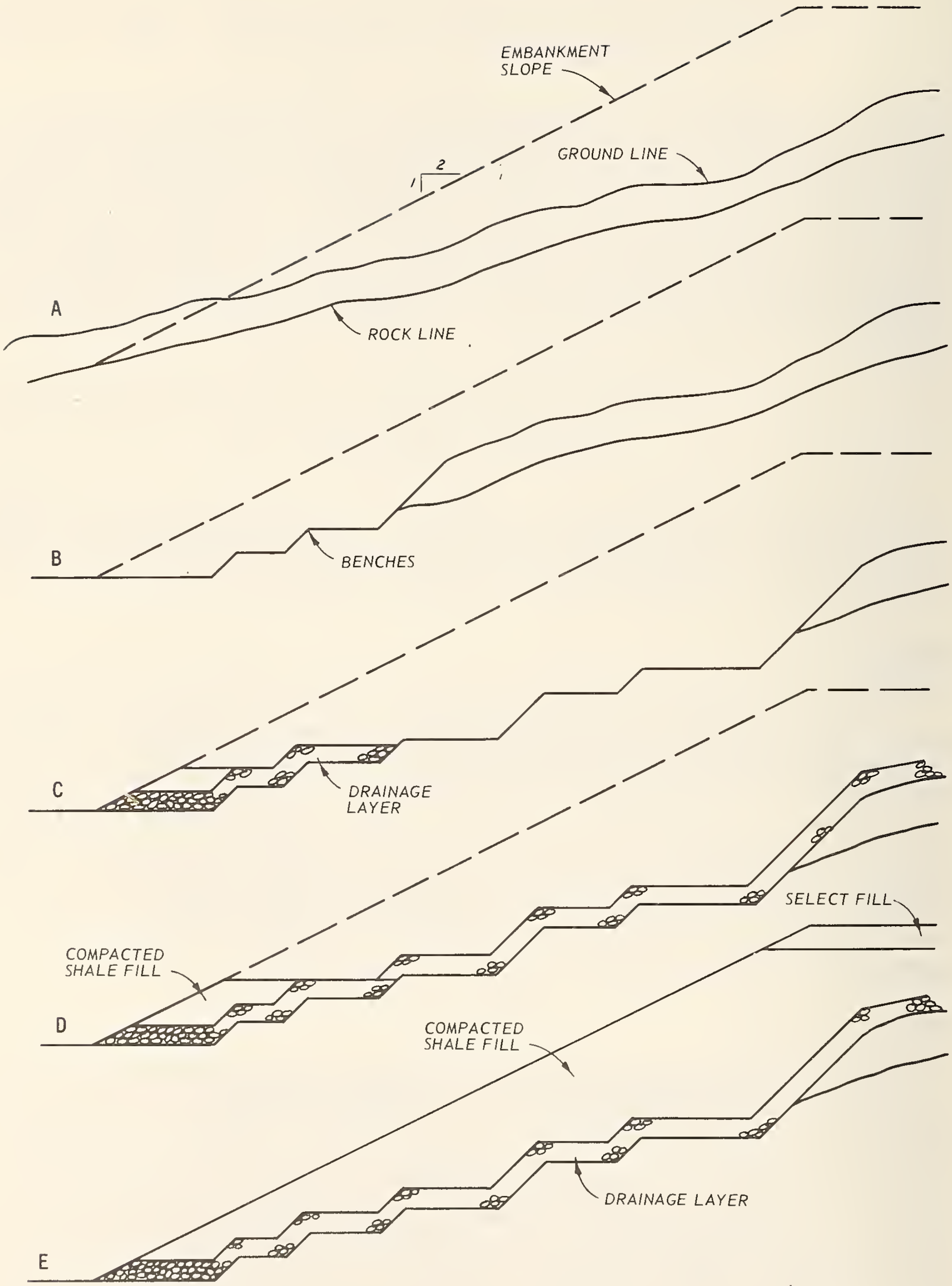

Figure 31. Construction sequence for benches and drainage layer

is the use of filter fabric (paragraph 107), which requires special handling and controlled placement to prevent damage during fill placement. Drainage layers must be protected after placement to prevent muddy water and fines from being washed into and clogging the rock. This problem can be minimized by requiring drainage layer placement to lag at least two benches below bench excavation (Figure 31B).

Bench drains

154. If there is not enough durable rock for a continuous drainage layer, then individual bench drains (Figure 21) should be installed. Bench drains to intercept seepage from permeable layers, such as jointed limestone or sandstone, require free-draining select aggregate or gravel with less than 10 to 15 percent fines. Collector pipes to conduct the water out of the embankment should be ditched across benches before embankment shale fill is placed and compacted. Collector (or outlet) pipes should not be placed higher in the fill, since they can be broken by subsequent settlement after construction. Outlets should be protected against freezing to ensure unrestricted water flow. If specific locations for drains are not shown on the plans and strata are seen to slope into the embankment foundation area, drains should be installed along benches intersecting limestone or sandstone stratum (Figure 21).

Underdrains and springdrains

155. It is particularly important for shale embankments that all potential seepage areas and springs in the embankment area be drained. Transverse slopes into cuts at the ends of cross valley or through embankments are especially important. *Every effort should be made to identify potential seepage areas during the design investigations.* However, during clearing of foundation areas for embankments, inspectors should look for previously unidentified wet spots, seeps, or springs. If these seepage conditions are found and not shown on the plans, additional drains should be installed or planned drains extended. Where graded aggregate and sand filter materials are scarce, filter fabrics have been allowed. Installation procedures are given in several references (see paragraphs 107 and 136). Filter fabrics are particularly useful for springdrains, since the fabric can be laid on the ground over the spring, coarse aggregate and the drainpipe and section placed on the fabric, and the fabric folded over the aggregate and drainpipe. An inspector's checklist should be made for drainage installations such as shown in Table 12. Construction manuals* also provide general guidance.

Excavation Procedures

156. Excavation procedures also must be tailored to the formation

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* West Virginia Department of Highways, Construction Manual, (Division 606 Underdrains), 1970.

Table 12. Inspector's Check List for Underdrains

1. Have all embankment foundation areas been checked for springs and seepage after heavy rains?
2. Is the drainage pipe placed and bedded in accordance with plans, specifications, and special provisions?
3. Have all special problems pertaining to drainage been brought to the attention of the project engineer?
4. Have an adequate number of outlets been provided for all underdrains? Are outlets so located that there is no chance of water backing into underdrains during heavy storms?
5. Have all pipe outlets been checked to insure they have not been crushed or displaced during construction?
6. Have all outlets been checked after periods of heavy rainfall to insure that they are flowing freely?
7. Has contractor placed markers at outlet end of all underdrain installations?
8. Are outlets protected against freezing to prevent ice buildup (northern climates)?

conditions in each cut to produce the required breakdown of nondurable thick shale strata or interbedded shale and harder rock. The degree of detailed information on formation characteristics (Part V), as shown on Geotechnical profiles and sections in the construction drawings, should serve as a basis for the required excavation procedures. The amount of ripping and blasting (paragraphs 96 and 100) will require trial and error, with the cooperation and expertise of the contractor, to obtain adequate breakdown or fragmentation.

157. In cuts of nearly horizontal thick shale and harder rock strata, each different stratum (classified as soillike or rocklike) should be ripped and/or blasted separately. This procedure will prevent mixing of durable rock with nondurable shale. However, durable (rocklike) shale and sandstone (or limestone) could be excavated together for rockfill sections of embankments. The main criteria is that nondurable shales, especially where interbedded in thin layers with other rock, must be broken down to meet size limits for compaction in thin lifts.

Field Classification of Shales

158. Three tests can be used to check the durability classification of shales during construction. The simplest is the jar-slake

test (paragraph 67), which can be performed in about 30 minutes by using a rapid means of drying out the shale pieces. A hot-air blow-dryer (300- to 500-watt heat blowgun) or microwave oven require about 15 to 20 minutes to dry out 1- to 2-in. shale pieces. The slake-durability test (paragraph 68) requires about 2 hours with a rapid drying method. The third test is the point-load test (Vol. 4), which is the fastest method, since a 2-in., irregular shale piece can be picked up and tested within 5 to 10 minutes. However, this test requires a previously established correlation with the jar-slake or slake-durability test for each major shale or siltstone strata within a particular cut. Consequently, it may be simpler to give descriptions of different nondurable shale, claystone, or siltstone strata on the geotechnical profiles or in preconstruction instructions to the project engineer. These descriptions would help the project engineer staff to identify the nondurable strata in the field during construction. Special instruction or training on the identification of nondurable shale materials also could be given to grade inspectors by a member of the geotechnical staff during test pad construction.

Selective Grading

159. The amount of selective grading depends on the thickness and inclination of different strata in a cut (Figure 1). The geotechnical profiles and typical sections should show the intended use of different strata as soilfill or rockfill (Figures 11 and 22a). Some stockpiling may be required initially unless soil, weathered shale, and nondurable shale from upper portions of cuts or from bench excavation can be placed directly in the central portion of a through (cross valley) embankment with a relatively level foundation. Rock stratum for use as rockfill drainage layers and in the outer sections of embankments can usually be routed directly to the proper location. Placing rockfill indiscriminately in the same lift with soillike shale or in separate lifts across the entire embankment should not be allowed since the rockfill can act as a reservoir for infiltrating surface or seepage water (Figures 4b and 51).

160. Selective grading may also be required to move large durable rock out of soillike shale during placement in thin lifts. A dozer equipped with the proper size rock rake can effectively push large stones to the outer slope. The danger is that large pieces of nondurable shale could be pushed along with durable rock. This danger can be minimized by first breaking down shale pieces with dozer treads or a heavy tamping roller equipped with small area square or pointed feet, as shown in Figure 32.

Compaction Equipment Capabilities

Soillike shales

161. Nondurable shales placed in thin lifts usually require two

Figure 32. Heavy tamping roller with square feet

different types of compaction equipment, based on experience with Indiana test pads (paragraph 120), shale embankments in Ohio (Vol. 4), and Corps of Engineers dams (Vol. 1). The compaction equipment used in these instances is listed in Table 13. Indiana test pad studies indicate that the required compaction (95 percent T-99) was not obtained with one compactor alone even where 12 coverages were applied (four-wheel compactor, dozer, Figure 33). Based on the above-cited experience, the type compactors, minimum weights, and number of coverages, as tabulated below, should be required.

Type Compactor	Minimum Roller Weight lb	Number of Coverages
Static, self-propelled or towed tamping foot roller (square or pointed feet for hard shales or shales containing limestone or sandstone)	53,000	2 to 4
Vibratory, self-propelled or towed	55,000 Compactive force	2 to 3
Towed, 4-wheel pneumatic-tired roller	100,000	2 to 4

To achieve adequate compaction of 8- to 10-in. loose lifts, the combined use of a tamping roller, followed by a vibratory roller or a tamping

Table 13. Examples of Shale Compaction Procedures and Equipment

Agency	Project	Shale Description	Selected Compaction Procedure	Compactors: Type	Weight: Total (lb)	Weight: At Drum (lb)	Centrifugal Force/Frequency (lb/vpm)	Dynamic Compactive Effort* (lb force)	Drum Rolling Width (in.)
Indiana State Highway Commission	Shale test pad R-Contract No. 9512 Perry County, IN, Report dated 3 Aug 78	Formation: Palestine; grey silty clay (shale) med. hard, flaggy. Soillike, I_D = 76 (avg) AASHTO Class: A-6 Opt w.c. = 10 to 13% Max γ_d = 122 to 126 pcf	10-in. loose lift, watered and disked; target w.c. of opt to opt - 2%. Four coverages of static roller, followed by three coverages of vibratory roller to obtain 100% of T-99	Static: Self propelled, dual drum with 12-in.^{2} tamping feet (Hyster C450A)	53,400	26,700	-- --	--	72
				Vibratory: Self propelled single drum with pads (Dynapac CA-25PD)	24,150	14,650	44,000/ 1700	58,650	80
Indiana State Highway Commission	Shale test pad R-Contract No. 10783 Orange County, IN, Report dated 8 May 78	Formation: Mansfield; dark grey silty clay (shale) soft, flaggy. Soillike, I_D = 27 AASHTO Class: A-6 Opt w.c. = 15% Max γ_d = 113 pcf	8-in. loose lift, watered and disked; target w.c. of opt to opt - 2%. Three coverages of static roller, followed by two coverages of vibratory roller to obtain 95% of T-99	Static: Self propelled, four drum tractor with 30-in.^{2} tamping feet (Cat. 825B)	64,800	16,200	-- --	--	40
				Vibratory: Self propelled single drum with pads (Raygo 420A, same as 420C except for size of pads)	25,586	15,560	32,000/ 1500	47,560	80
Ohio Department of Transportation	I-74 at S.R. 128 Hamilton County, OH, (see Vol. 4)	Formation: Kope; grey clay shale inter-bedded with limestone. Soillike, I_D = 26 (avg) AASHTO Class: A-6 Opt w.c. = 11% Max γ_d = 122 pcf	8-in. loose lift, four coverages of fully ballasted tamping roller and two coverages of 50-ton pneumatic roller	--	--	--	-- --	--	--
Corps of Engineers	East Fork Lake Dam	Formation: Kope; grey clay shale inter-bedded with limestone, I_D = 70, soillike	8-in. loose lift, two coverages of tamping roller and four coverages of 50-ton pneumatic roller to obtain 95% compaction (T-99)	Tamping roller: Self propelled, dual drum with 6-in.^{2} square feet (Hyster 455A)	51,650	25,800	-- --	--	72
				50-ton: towed, four wheels abreast, 80 to 100 psi tire pressure	100,000	25,000 per wheel	-- --	--	--

* Centrifugal force plus static drum weight.

Figure 33. Four-wheel compactor dozer

roller, followed by a pneumatic tired roller and a minimum total of six coverages may be necessary. The speed of compactors should not exceed 3 to 5 mph. Test pads should be used to determine the optimum number of coverages with each compactor where previous experience is lacking with a particular shale type.

Rocklike shales

162. For durable shales placed as rockfill in loose lifts not exceeding 24 in. maximum (18 in. for bridge approach embankments where piling will not be driven), 10- to 15-ton vibratory compactors have proven satisfactory (Vol. 1, Table 12). For clean rockfill, hauling and spreading equipment, when routed uniformly over each lift, may be adequate. Rocklike shales in which the amount of soil or fines cannot be controlled should be limited to 12- to 18-in. loose lifts and compacted using heavy (10- to 15-ton) vibratory or pneumatic-tired (50-ton) rollers.

Test Pad Construction and Testing

163. Test pads should be constructed at the start of and during

construction when experience is lacking on compaction of nondurable shales from a particular formation. Details for test pad construction and testing are given in Appendix A and are discussed briefly below.

Purpose

164. Test pads are constructed to determine the following:

- a. Applicability of watering and disking in breaking down shales and improving compaction after placement.
- b. Developing the best procedure for breaking down oversize shale and rock or raking durable rock out of nondurable shales.
- c. Suitability of the contractor's compaction equipment.
- d. Optimum compaction procedure for obtaining desired degree of compaction.

Minimum requirements

165. At least three test pads (about 200 by 40 ft each) are needed to vary the number of coverages from two to four for the static compactor and then two to four coverages on each pad for the second compactor (vibratory tamping roller or 50-ton pneumatic roller). A heavy static compactor with square or pointed feet (Figure 32) may be required to achieve further breakdown of hard nondurable shales and shales containing numerous chunks of limestone or sandstone that cannot be raked out of the loose layer.

166. Test pads need to be constructed on a level, firm base that will not settle. Dry shales that slake appreciably (Figure 12, $I_J \geq 3$) should be watered from a spray bar after placement and worked with a heavy-duty disk (36-in.-diameter) to obtain as uniform a mixture as possible near optimum water content. A three-point compaction test on minus 3/4-in. material (paragraph 85) should serve as a comparision for percent compaction. After each compactor coverage, three to five in-place density tests using two methods such as the 6-in. sand cone with the speedy moisture meter and direct transmission nuclear moisture-density apparatus are needed to determine density increase versus number of compactor coverages. The sand cone and speedy moisture meter methods are necessary to check the calibration of nuclear moisture-density apparatus (see Appendix A).

167. Once the compaction procedure is established, inspection can be accomplished by measuring loose-lift thickness, noting maximum rock size, and counting the number of coverages by each compactor. Occasional in-place density and one-point compaction tests may be desirable to verify that the desired percent compaction is being obtained.

Compaction Procedures and Control Techniques

168. Compaction procedures for shale embankments depend on the degree of compaction required by settlement limitations. For minimum settlement, several steps may be required prior to rolling to obtain additional breakdown after placement, especially for hard, nondurable shales and softer shales in thin layers interbedded with limestone layers. Control techniques for procedural-type special provisions involve inspection to verify or enforce lift thickness and criteria on maximum rock size, amount of oversize pieces, percent of fines, adding water, disking, type and weight of compactors, and number of coverages. End result specifications, based on a required percent compaction (or percent relative compaction) are more difficult to enforce since considerable in situ density and field compaction testing is required.

Compaction procedures

169. For minimum settlement cases, the following procedures may be necessary to achieve adequate compaction. Nondurable shales are generally required to be spread in loose lifts not exceeding 8 in. with oversize pieces broken down or removed. Four-wheeled, heavy (30-ton) compactor dozers with flat or low-angle-wedge pads (Figure 33) are often used to spread and compact, but tend to ride over rather than break down hard shale, limestone, or sandstone chunks and slabs. Heavy, tracked dozers followed by compactors with small area square feet (Figure 32) are more effective. Initial rolling followed by watering of dry shales (using spray bar-equipped tankers or trucks) can also help in breaking down oversize shale pieces. Dry shales that slake readily should be watered and disked (36-in. disk) to aid in compaction. Water added should not increase the in situ water content above optimum for the shale. Initially, about 1 gal/loose yd (6-ft-square section of 8-in. loose lift) can be tried. A dense layer should be produced by the following procedure: disking, followed by additional watering (if no "gummy" clods are apparent), then static roller compaction with a minimum of two complete coverages, followed by vibratory roller compaction to bring the total number of coverages to six. The above procedures can be modified on the basis of test pad results and by the use of thicker lifts for less stringent requirements on long-term settlements.

Poor practices

170. The following practices often lead to problems after construction and should be avoided:

<u>a</u>. Haul roads of uncompacted material are often incorporated into the embankment. All haul roads within the shale embankment area should be compacted to the same degree as the embankment. This requirement applies especially to haul roads along the side of an embankment.

b. Lack of compaction often occurs at the outer edge of embankments or at the ends of fills adjacent to slopes coming up into cuts. This problem can be aggravated by the requirement to move oversize rocks to the outer slope. Large shale chunks, clods of soil, and trash are easily included to form a soft outer zone that washes or slumps after construction. Outer rock zones of shale embankments must be kept free of loose soil and trash. Outer zones of shale fill must be well compacted.

c. Lack of benching often occurs at the end of a previously constructed section of embankment (such as a bridge approach fill) when the remainder of the embankment is constructed. Benching of the old slope to properly "key" in the new embankment is necessary to prevent a loose soft zone where the two embankments meet.

d. "Double lifting" is common (placing next lift before the last lift is properly compacted), especially where 8-in. loose lifts are required. Frequent inspection, checking by driving a reinforcing bar into the underlying lift, or excavating an inspection hole through the top lift may be necessary.

e. Lack of an adequate number of compactors is a major problem in shale embankment construction. For example, compaction of a 200-ft-long, 66-ft-wide area with three coverages by a static roller, followed with three coverages by a vibratory roller operating at 3 to 5 mph requires at least 25 minutes. Four 20-yd scrapers with a 5-minute turnaround time can place 80 yd in 7 minutes or a 200- by 66-ft area in 30 minutes. Thus, a new area for compaction could be dumped and spread by dozers almost as fast as the time for compaction. If a compactor broke down, or a fifth hauling unit were added, or the turnaround time shortened, then the compaction would lag behind and a double lift could be placed more than once during the shift. Thus, the inspector must observe the operation long enough to see that correct procedures are followed and check during the shift to observe and report changes that prevent complete compaction.

Compaction control techniques

171. Procedural provisions. In addition to determining that the proper ratio of hauling and compaction equipment is being used, inspection to enforce procedural provisions for soillike shales includes the following:

a. Specified lift thickness. Hand-level measurement of change in height of fill surface after lift placement or after compaction using several 7-ft laths (marked at 8-in. intervals above and below eye level) hammered into the edge of the fill. Alternatives include (a) pushing a marked reinforcing bar or narrow shovel into loose fill to find previously compacted surface or (b) driving a stake at the fill edge, marking 8 in. above the last lift layer, and then standing downslope to compare the new fill surface line with the mark on the stake, using a hand level.

b. Oversize rock. Detected mainly by observation and judgment, supplemented with size measurements.

c. Allowable percent oversize rock. Determined by observation and judgment to see that 6 in. or larger size rock is less than 20 percent (not more than seven 6-in. or larger pieces in a square-yard area).

d. Requirements for watering and disking. Determined by slaking of dry shale pieces (2- to 3-in. size) in water and excessive amount of shale chunks in layer. Uniform application of water by spray bar attachment at rate of 1 to 2 gal/yd, followed by disking with heavy-duty disk (36-in.-diameter disk) pulled by a heavy tractor.

e. Suitability of compaction equipment. Require certified copy of manufacturer's specification data sheet for each compactor, inspect operating condition, and check technical data plate on unit, type, and size of roller feet, and type and amount of added ballast.

f. Compaction. Count number of coverages, clock operating speed (3 to 5 mph), and check condition of layer after compaction (dense, well compacted with no significant voids or loose pockets of material).

172. End result provisions. For soillike shales, in-place density data are required to determine that the specified percent compaction is being obtained. Normal procedures are used for shales containing not more than 35 percent plus 3/4-in. size. For shales containing more than 35 percent plus 3/4-in. size, the following procedure is applicable:

a. In-place density. For soillike shales, experience in Indiana indicates that properly calibrated direct transmission nuclear moisture-density gages (see Appendix A) produced results comparable to the 6-in. sand cone method and speedy moisture method.

b. Determining percent compaction. Because of the wide variability in shale gradation and compaction within a single lift, a standard set of compaction curves cannot be used. In-place density should be compared directly with one-point compaction tests on two to three samples (taken prior to compaction) from the lift being tested. Samples should be screened to remove plus 3/4-in. sizes and one-point tests performed on minus 3/4-in. material in a 6-in.-diameter mold. Attempting to model the fill material gradation or using excavated material after field compaction is time-consuming and can produce misleading results because of additional breakdown during excavation and sieving.

Control of durable shales in rockfill

173. Rocklike shales used in rockfill should be restricted to 24-in. maximum lifts. The main control is preventing excessive fines (greater than 20 percent minus No. 4 sieve material, paragraph 117). If the amount of fines cannot be controlled, thinner lifts 12 to 18 in. and compaction by heavy rollers should be required to prevent rock from being surrounded by loose soil or fines. Three to six coverages with heavy vibratory rollers (10- to 15-ton total weight) and/or 50-ton pneumatic tired rollers (Table 13) are necessary to achieve adequate compaction. The same techniques listed for procedural provisions (paragraph 171) apply.

Berms, Buttresses, and Retaining Structures

174. Construction of special design features (stability improvement features) for shale embankments must insure plans and specifications are followed. Proper drainage and adequate compaction are critical items that require frequent inspection.

Berms and stabilization trenches

175. Berms added to the downhill side of shale embankments for stability must be benched into unweathered material on slopes and have adequate foundation drainage. The same control criteria apply for benching, drainage, and compaction as for the main embankment described in the previous section. Berms placed and compacted after the main embankment must be benched into the main embankment. Stabilization (or shear) trenches are specified to handle seepage in deep hillside deposits of weak soils (Part IX).

Rock buttresses and pads

176. Rock buttresses and rock pads at the downhill side of shale embankments (Part IX) must be composed of free-draining durable rock

(Royster, 1975) and should be constructed at the same rate as the main embankment. This requirement is necessary to maintain a level section and prevent fines from washing into the rock. Requirements for filter material or fabric between the shale embankment and rock pad or buttress to prevent internal seepage erosion of shale and clogging of rock is described in Part IX, paragraph 242. Horizontal drains are often required where excess seepage is present. The drain spacing (or location) and slope as shown by the plans must be strictly followed. Horizontal drains installed during construction can be wrapped with filter fabric and should be laid in a shallow V-shaped trench and covered with sand or gravel to prevent crushing during placement and compaction of the next fill layer. Horizontal drains should be extended through the rock to the outside slope to provide access after construction for inspection and cleaning. Horizontal drains installed after the embankment and buttress are completed requires special horizontal boring equipment.

Retaining structures

177. Retaining structures usually include gabion walls, reinforced earth walls, or crib walls on the downhill side of shale embankments. These structures require excavation into unweathered material for a stable foundation. Construction of gabion walls (Blackburn, 1973)* requires durable, free-draining rock placed in a compact arrangement to prevent future settlement within the basket. Filter stone and/or fabric is required behind the wall to prevent seepage water from eroding fines out of the embankment and clogging the rock (Part IX, paragraph 242) and causing failure (Blackburn, 1973). Reinforced earth walls (Trolinger, 1975)** require special granular backfill and construction procedures discussed in Part IX. Crib walls of concrete or metal struts require free-draining, dense granular or stone fill and filter material or fabric behind the wall to prevent clogging of the wall fill (Part IX). Another important item for gabion and crib walls is to prevent soil from being dumped or washed onto the final surface of the rock or stone fill. The soil could eventually infiltrate and clog the rock or stone fill.

Instrumentation Installation and Observation

178. Installation of instruments in major shale embankments

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* Blackburn, J., "Gabion Construction on Slides at Interstate 40 Near Rockwood, Tennessee," Proceedings of the 54th Annual Tennessee Highway Conference, Bulletin No. 39, Engineering Experiment Station The University of Tennessee, Knoxville, TN, Jan 1973.

** Trolinger, W. D., "Construction of the I-40 Reinforced Earth Embankment," Proceedings of the 56th Annual Tennessee Highway Conference, Bulletin No. 41, Engineering Experiment Station, The University of Tennessee, Knoxville, TN, Jan 1975.

requires experienced technicians and should be supervised by a member of the geotechnical staff or an experienced project engineer. Generally, it is more economical to install casing for slope indicators and piezometers (or open casing for groundwater monitoring) after construction. However, devices to measure vertical movement or compression during construction and after need to be installed during construction. Surface settlement and lateral-movement points required on the outer slopes can also be installed during construction (when the embankment has reached a height where equipment will not damage the completed installation). Types of instruments are described in Synthesis of Highway Practice 8, "Construction of Embankments" (Highway Research Board, 1971) and in the sources referenced in paragraph 144.

Vertical movement devices

179. Installation of vertical movement measurement devices, such as the U. S. Bureau of Reclamation (USBR) cross-arm type with telescoping riser pipes, can be installed to provide measurements at 10-ft intervals at one location. Procedures for installation in rocky soils are given in the USBR's Earth Manual (Second Edition, 1974). Other types of single settlement plates with a riser pipe can be installed using the same general procedure as for the USBR cross-arm device. The main consideration is to protect the riser pipe sections from damage during placement and compaction of the fill. Protection is usually accomplished by placing 3- to 4-ft-high mound of soil around the exposed riser pipe and marking the pipe location with a large flag on a stick extending several feet above the pipe.

Observations

180. During construction, observations of vertical movement devices and surface settlement and lateral-movement points on the outer slopes should be made weekly or for each 10 ft of height increase. Observation procedures are described in the references of paragraphs 178 and 179. The purpose of observations during construction is to determine the rate of vertical compression and lateral slope movement of the fill with time. Lateral movement observations can indicate distress or danger of a slide occurring during construction. Data obtained during construction can provide a valuable basis for evaluating observations after construction.

Construction Records

181. Construction records containing specific information on the following items for shale embankments can provide valuable data. The data, when compared with long-term service performance (settlement and slope stability), can lead to selection of optimum requirements for drainage measures, compaction procedures, and control techniques for future construction with improved economy:

a. Benching and drainage installation locations, procedures, and drainage materials used.

b. Excavation methods, equipment, and procedures.

c. Copy of special provisions.

d. Extent of selective placement and locations for different materials.

e. Equipment and procedures used in processing shale after placement (adding water, disking, and removing or breaking down oversize shale or hard rock).

f. Compaction provisions (end result or procedural).

g. Loose-lift thickness.

h. Compaction equipment and procedures.

i. Control procedures including tests to check shale durability, summary reports on test pads, location of in-place moisture-density tests, procedures for correlation with maximum density and optimum water content, and summary of test results and percent compaction obtained (for end result specifications on soillike shales).

j. Complete information on procedures and drainage measures used for construction of shear trenches, reinforced earth, gabion or crib walls, and other special stabilizing measures.

k. Photographs of various phases of construction including compaction equipment in operation and installation of drainage measures.

Complete information in construction records reduces the time and costs for determining the source and cause of distress that might occur several years after construction. Complete information also provides a sound basis for modifying required construction procedures for future shale embankments.

182. The grade inspector's daily diary is the most important record, since it contains detailed information on all phases of the work. For shale embankments, as much of the information as possible listed in Table 14 should be included initially with changes noted as they occur. If standard forms are used instead of field notebooks, a standard list of needed information on compaction equipment could be printed on the back or a standard supplemental form used. Information on blasting used to break down shale materials can provide future data

Table 14. Important Information Needed in Grade Inspector's Records for Compacted Shale Embankments

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A. Excavation and hauling operations:

1. Stations and approximate elevation.

2. Type excavation (e.g., selective by different material layers or excavated as one unit).

3. Natural moisture content of shales, siltstones, etc. (e.g., estimated as dry, moist, wet, or give measured value).

4. Type material (e.g., hard, bedded shale; interbedded shale (percent) and limestone (percent); siltstone; weathered shale, etc.).

5. Method of excavation and procedure (e.g., scraper pushed with D-9; ripping and cross-ripping to 2-ft depth; blasting, include blasting report, Figure 34).

6. Procedure used to break shale and harder rock into small pieces (e.g., tracking with D-9; extra blasting; powered impact hammer).

7. Description of excavation equipment:

   Number of each type.
   Make and model including size and number of ripper teeth on each ripping unit.

8. Hauling equipment:

   Number of each type.
   Make, model, capacity.
   How loaded (e.g., down 10 percent grade; by pusher dozer; no assistance).
   Load (e.g., heaping; full; 3/4 full; etc.).

9. Haul roads:

   Amount of watering and compaction when fill is used for haul roads within embankment area.
   Routing of hauling equipment on embankment fill (e.g., staggered across the fill or using same path).

B. Bench excavation:

1. Stationing and approximate elevation.

2. Excavation method (e.g., scrapers; ripping; blasting).

3. Depth and width of completed benches.

4. Type material excavated (e.g., weathered shale; siltstone; residual soil with limestone rocks; etc.).

5. Material at bench bottom (e.g., hard shale; limestone layer; unweathered siltstone; etc.).

Table 14. Important Information Needed in Grade Inspector's Records for Compacted Shale Embankments (Continued).

C. Method for checking durability classification of shale, siltstone, claystone, etc. for compaction as soil or use as rockfill or drainage layer (type test and results).

D. Fill placement and processing:

1. Stations and elevation.

2. Number of hauling units and round-trip time.

3. Source and description of fill material (e.g., freshly blasted hard durable shale and siltstone from station ______ to ______, 12-in. maximum size, 20 percent plus 6 in., 30 percent limestone to 2-ft slabs mixed with soillike shale, etc.).

4. Spreading equipment (e.g., D-9 dozer compactor with blade, etc.).

5. Equipment and procedures for breaking down shale or removing oversize rock pieces (e.g., D-9 dozer; roller with pointed feet; limestone over 1 ft pushed to outer slope).

6. Method of adding and mixing in water for dry shale (equipment type, make, and model; type spray bar attachment; size disks on disking equipment, type towing equipment) and method of checking for proper moisture content.

7. Special treatment measures such as disking in lime to neutralize acid shales.

E. Fill Compaction:

1. Loose-lift thickness and method of checking.

2. Compaction equipment:

   Number of each type.
   Make and model.
   Number of drums and size (diameter and rolling width).
   Size of tamping feet or pads (length of shank and tip area or contact area).
   Weight of roller as used.
   Type of ballast.
   Type frame (rigid or oscillating).
   Speed of travel during compaction.

   For vibratory rollers include:

   Static weight of vibrating drum, operating frequency, and rated centrifugal force.

   For pneumatic rollers include:

   Number of boxes or sections, overall width and length, number of wheels, wheel spacing, tire size and pressure, and weight per wheel as used (or total weight).

Table 14. Important Information Needed in Grade Inspector's Records for Compacted Shale Embankments (Concluded).

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3. Number of coverages of each type compactor.
4. Method of checking adequacy of compaction for procedural provisions.
5. Method of checking fill water content and percent compaction (or relative compaction) for end result specifications and for relating in-place results with appropriate maximum density and optimum water content; also number of in-place tests per shift (or layer or by volume).
6. Treatment of wet shale (e.g., muddy shale and soil scraped off and wasted; dry shale disked in with wet shale).

F. Maintaining uniform fill surface and compacting outer slopes:

1. Whether fill is being brought up uniformly or in sections (e.g., outer rockfill section is 5 ft below central shale fill, etc.).
2. Surface (is) or (is not) being sealed and graded to drain overnight.
3. Whether outer slope of shale is being compacted by rollers or only by dozer tracking up and down the fill slope.

G. Problems encountered and actions taken on the following:

1. Breaking down shale and rock during excavation (inadequate ripping or blast holes too far apart, etc.) or during compaction.
2. Checking durability classification of shale, siltstone, etc.
3. Excessive amount of large rock in shale fill or too many fines in rockfill, etc.
4. Adding and mixing in water for dry shale and adequacy of compaction (spray bar not being used, disking equipment too lightweight, too few compaction machines, etc.).

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for a particular formation. A standard form used by Montana is shown in Figure 34. A brief hypothetical example of a daily diary is shown in Figure 35.

BLASTING RECORD

Project No.

Date Time

Area of the Blast

Station

Station

*Formation Name:

*Description:

Primary Holes

Number of holes *Pattern

Hole length Diameter *Subdepth

Hole spacing by

Pounds powder / yd^3

Type of powder

How loaded

Type of stemming stemmed ____ ft. from surface

Primer type

Primer weight

Number of ignition delays

Ignition: Electric ☐ Fuse ☐ Primacord ☐

Comments on fragmentation, backslop condition before and after blast and any general comments.

*Range of Max Size: *Approx. % minus 3 in.:

FORM NO. CSN 55

HOLES Page No.

Presplitting Record Lift No.

Number of holes

Hole length Diameter

Hole attitude N S E W

Hole spacing

Pounds powder / linear foot How Loaded

Powder type

Stemming

Position

Station to

How ignited ☐ Electric ☐ Fuse ☐ Primacord

Distance to closest primary hole

Sketch showing primary and presplit ignition sequence, position of free face and back slope, direction of hole spacing

	Date
Entered By:	
Figured By:	
Checked By:	

FORM CSN

Note: * added for shale formations.

Figure 34. Example of blasting record form (courtesy Montana State Highway Commission)

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Project: SP-19-261
10 Jul 78, 6:00am - 2:30 pm shift
Inspector: Ira M. Tough
Contractor: J. Brown Co.
Bench excav. - sta 976-984
moist weath. gray shale & brown
siltstone. Equipment:
1 - CAT DD9H w/ripper
2 - CLARK 210 scrapers
Excavating 2-3 ft to good unweathered
material on upper levels. Placing
sandstone drainage pad on lower
levels.
Excav. mt'l from sta 976-984 to
upper part of fill sta 955-960
(el 1275, 45-ft height). Equipment
on fill:
1 - CAT 825B comp. dozer
1 - RAYGO 420 C
1 - Water truck IH w/spray bar
Weath. shale & siltstone (6-in. max.
size) in 10-in. lifts using 825B
w/3 coverages then 3 cov. w/420C.
Breaks down to -2 in. No water
required.

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Sta 956+50 and 958+00, el 1276:
Two density tests = 96% comp. (based
on 1-pt. T-99 on -3/4 in.).
Excav. hard shale and sandstone from
cut, sta 920-931; blasted el 1310 to
1304 last shift change. Max. size =
18 in. w/15% minus 1 in. Two TEREX
T532 scrapers hauling to outer rock-
fill sec. at sta 931-940 and base
pad sta 912 to 918. Two-ft layers
compacted by 3 coverages BW 210D vib.
roller. Soaking test on oven-dry
shale - no slaking or cracking - OK for
rockfill.

1:00 pm - 30-min. rain.
Fill at sta 955-960 too wet. Con-
tractor to bring in disk to rework,
el 1277.
2:10 pm - stopped contractor -
sta 955-960 until disk is on fill
and material dried back. Informed
project engineer.

Figure 35. Hypothetical example of an inspector's diary for one day

PART VIII: EVALUATION OF SHALE EMBANKMENTS

Identification of Potential Problem Embankments

183. Large highway shale embankment failures have typically been preceded by settlement, cracking of the pavement, and surface slides on embankment slopes (Vol. 2). To detect probable failures, maintenance records should be periodically reviewed by the geotechnical staff. This review should identify areas of repeated overlays, the location of pavement cracking, and areas where slope repairs have been made. Geotechnical personnel should log the location and nature of such repairs and when the rate of recurrence of pavement overlaying, crack repairs, or repair of surface slides becomes excessive, and evaluation of the embankment should be undertaken. When a number of embankments are exhibiting distress, it may be necessary to assign priorities and establish an investigation program in order of priority and available funds.

Definition of Cause

184. Defining the cause of distress or failure requires the compilation and reviewing of all available information to establish factors that influence the embankment behavior. This task is normally undertaken by geotechnical personnel and involves the collection of site data related to:

- a. Soils and geology.
- b. Groundwater levels and flow patterns.
- c. Design criteria and construction methods.
- d. Embankment behavior.
- e. Embankment and foundation material strengths.

Historical review

185. A certain portion of available data is historical and includes preconstruction and postconstruction information. The amount and types of historical data available will depend upon the extent of the predesign investigations and the monitoring conducted during and after construction. Information gathered during planning and route selection might include:

- a. Topographic base maps of the area.
- b. Agricultural soils maps.

c. Geologic literature.

d. USGS water resources reports.

e. Test borings and soil test data from adjacent local projects.

f. Aerial photographs (mosaics, stereo pairs, color photography, and remote sensing).

g. Corridor study reports.

h. Field reconnaissance reports.

i. Route selection investigation reports.

186. Additional information gathered during the design should also be reviewed. This data may be presented in various forms such as:

a. Materials reports, materials information reports, or foundation reports.

b. Soils reports.

c. Geophysical profiles or reports.

d. Rock outcrop maps.

e. Terrain reconnaissance reports.

f. Geologic profiles and sections.

g. Plans and specifications.

These sources will either contain plans and sections of the embankment, showing type of materials and stratification, or will greatly aid in the preparation of profiles and sections. The soils boring logs and laboratory data needed for the review will also be found in these reports.

187. Construction records should also be examined to determine the construction methods employed, the location of different types of material (shale, weathered shale, and soil, rock, etc.) within the embankment, the compaction procedures and control techniques used, and the problems encountered. Instrumentation data installed for construction and postconstruction monitoring should also be reviewed. The collection and assimilation of historical data may take place prior to or after the first site inspection has been made, depending upon the urgency of the embankment problem and the availability of the data.

Site inspections

188. The primary purpose of a site inspection is to determine the class of the embankment problem. Problems can be classified as:

a. Very serious (immediate repairs necessary with investigations to determine design parameters).

b. Serious (extensive investigation needed to determine cause and repairs needed).

c. Medium (possibility that problem might become serious; limited investigation with follow-up inspections).

d. Minor (maintenance repairs only).

Since site inspections are conducted in response to the first signs of distress, they will most likely be conducted with a minimum amount of equipment or with none at all. A sketch pad, a metal or cloth tape, hand level, compass, and a camera would be useful aids during the inspection. Geotechnical personnel conducting site inspections should:

a. Examine the area where distress was first noted and then examine all other portions of the embankment, including the slopes, roadway, median, shoulders, and adjacent embankment sections.

b. Make notes concerning the location of all of the following:

(1) Pavement and shoulder cracking.

(2) Sloughing.

(3) Seepage.

(4) Ponding.

(5) Spring areas.

(6) Drainage.

(7) Settlement of the embankment.

(8) Cracking of the embankment.

(9) Alignment of guardrail posts, signposts, poles, fences and trees.

c. Prepare a sketch showing locations of items in b and including the district, county, route, and post mile;

job number; limits of sketch; date; sketched by; north arrow; and direction of center line to known landmark.

d. Classify the problem.

e. Make recommendations for action, such as:

(1) Designs for immediate repairs can be made using strengths based on experience with similar materials and geologic conditions. Install instrumentation to monitor behavior.

(2) Monitoring behavior and making borings to determine in situ materials and properties with design of remedial measures being accomplished after analyzing the data.

(3) Maintenance repairs and behavioral monitoring with other action being taken only if the problem becomes more serious.

(4) Maintenance repairs only.

Collection of Behavioral Data

189. A complete description of the problem includes a record of embankment behavior with time. This normally requires the installation and monitoring of devices to determine groundwater level or pore pressure, surface displacements, and subsurface movements. In addition, seepage and/or discharge from drains should also be monitored. The following paragraphs discuss the devices and methods recommended for evaluation of shale embankments.

Piezometers

190. A piezometer is used to determine groundwater levels or pore-water pressures within a medium. This information is needed in the evaluation of the stability of a shale embankment. In addition, piezometer observations, in certain cases, may be used to determine permeability or to indicate the efficiency of a drainage layer beneath the fill. Selection of a piezometer should be based upon cost, durability, reliability, ease of installation, site conditions, expected fluctuation of the water table, frequency of observations, and response time.

191. A major consideration in selecting the proper type of piezometer for measuring pore-water pressures is the permeability of the medium in which the pore-water pressure is to be measured. The response time of a piezometer is dependent upon the permeability and the type of

piezometer installed. Response time is perhaps the most important consideration from the standpoint of the data collection. If the installed piezometer does not respond quickly enough to changes in groundwater conditions, the data collected will probably not reflect field conditions at the time of the observation.

192. Procedures have been outlined for determining the response time (EM-1110-2-1908; Terzaghi and Peck, 1967*); the procedure basically consists of either filling with water or emptying the piezometer and measuring the time it takes to approach its initial equilibrium condition. This is done by measuring the change in piezometric level at different time intervals. From these data, the basic time lag can be determined. The exact value of the time lag is not as important in the evaluation of the fill as the knowledge of the drainage characteristics of the fill obtained from such a test. Even though equations have been presented for determining permeability from piezometer readings (EM 1110-2-1908), the knowledge that the fill is free-draining or relatively impermeable is important in an analysis.

193. Figure 36 shows a plot of response time versus permeability

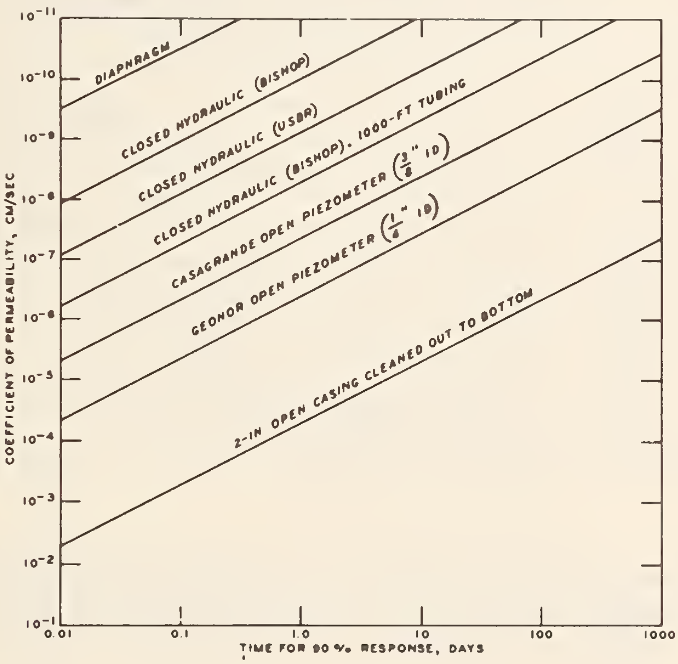

Figure 36. Approximate response time for various types of piezometers (after Terzaghi and Peck, 1967)

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* Terzaghi, K. and Peck, R. B., Soil Mechanics In Engineering Practice, 2d ed., John Wiley, New York, 1967.

for various types of piezometers. Permeability tests on well compacted samples (equivalent to 100 percent of AASHTO T-99 maximum density) of shale and siltstone from highway embankment locations (Vol. 4) indicate permeabilities on the order of 10^{-4} to 10^{-7} cm/sec. Considering these permeability estimates as an upper limit and the open-system piezometer as the simplest, least expensive, and easiest to install, the open-system piezometer is recommended for use in compacted shale embankments. The Casagrande open-standpipe porous-tube piezometer and the wellpoint piezometer are most frequently used (Figure 37). Also, a simple slotted or drilled pipe is sometimes used. For cases where soft layers of low permeability are encountered, a closed-system type piezometer may be required.

194. Information on different types of piezometers and guidance on selection, installation, and use in measuring groundwater levels and pore water pressures are given in EM 1110-2-1908. The three basic types of piezometers are open-system piezometers, hydraulic or closed-system piezometers, and diaphragm piezometers. Table 15 compares the advantages and disadvantages of these three types. A list of piezometers commonly used is shown in Table 16.

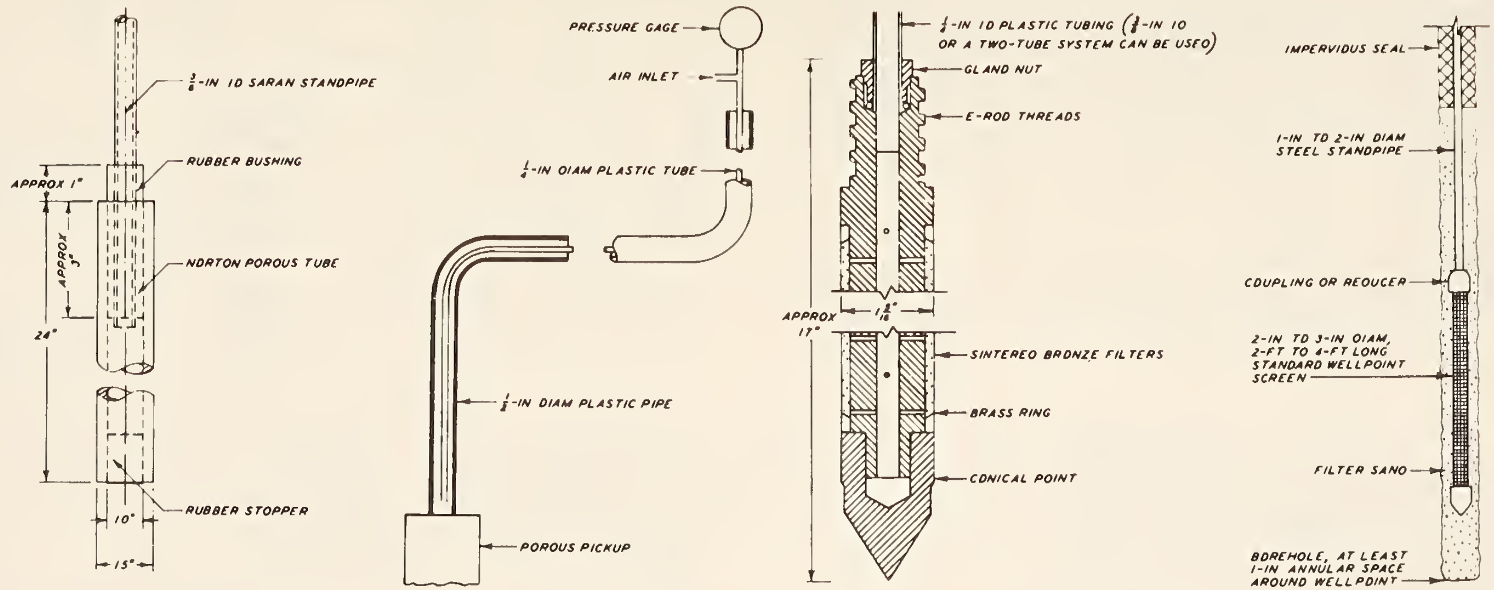
195. A typical embankment instrumented with piezometers is shown in Figure 38. Plotting of piezometer elevations versus time along with rainfall data or local stream flow data may help pinpoint the source of water entering a fill. Examination of the readings will indicate a well drained fill if readings for embankment piezometers 1 and 3 are always near the elevation of the top of the drainage blanket or if they are always dry. Piezometers 2 and 4 will reflect groundwater elevations within the foundation. A fill with poor drainage will conversely be indicated if piezometers 1 and 3 indicated piezometric elevations within the embankment and foundation piezometers 2 and 4 also indicated elevations within or above the embankment. In such a case, the embankment will have actually blocked a natural drainage pattern, a condition long recognized as potentially dangerous. Further indications of poor drainage can be seen if the water-surface elevations in piezometers 1 and 3 increased within the embankment following periods of heavy rainfall and took an extended time to regain their normal levels after the rainfall had ceased.

196. The frequency of readings obtained from a system of piezometers will depend upon a number of factors and will most likely be altered many times before a suitable interval is obtained. Some of the factors affecting the frequency of reading are:

<u>a</u>. Rainfall, stream fluctuation, or groundwater changes.

<u>b</u>. Magnitude of problems under study.

<u>c</u>. Initial evaluation of water surface at the time of installation.



a. CASAGRANDE POROUS TUBE

b. TWIN-TUBE CASAGRANDE (AFTER DE LUCCIA, 1958)

c. GEONOR PIEZOMETER

d. WELLPOINT

Figure 37. Open-system piezometers as given in EM 1110-2-1908

Table 15. Comparison of Piezometer Types as Given in EM 1110-2-1908

Basic Type	Advantages	Disadvantages
Open-system	Simple; comparatively inexpensive; generally not subject to freezing; relatively long life; fairly easy to install; long history of effective operation.	Long time lag in impervious soils; cannot measure negative pore pressure; cannot be used in areas subject to inundation unless offset standpipe is used; must be guarded during construction; no central observation station is possible; requires sounding probe.
Closed-system	Small time lag in any soil; can measure negative pore pressures; can be used in areas subject to inundation; comparatively little interference with construction; can be read at central observation stations.	Observation station must be protected against freezing; fairly difficult to install; fairly expensive compared to open systems; sometimes difficult to maintain an air-free system; most types are fragile; some types have limited service behavior records.
Diaphragm	Simple to operate; elevation of observation station is independent of elevation of piezometer tip; no protection against freezing required; no de-airing required; very small time lag.	Limited performance data, some unsatisfactory experience; some makes are expensive and require expensive readout devices; fragile and requires careful handling during installation.
	<u>Pneumatic.</u> Electrical source not required; tip and readout devices are less expensive than for electrical diaphragm types.	Often difficult to detect when escape of gas starts; negative pressures cannot be measured; condensation of moisture occurs in cell unless dry gas is used; requires careful application of gas pressure during observation to avoid damage to cell.
	<u>Electrical.</u> Negative pressures can be measured.	Devices subject to full and partial short-circuits and repairs to conductors introduce errors; some makes require temperature compensation and have problems with zero drift of strain gauges; resistance and stray currents in long conductors are a problem in some makes.

Table 16. Commonly Used Piezometers as Given in EM 1110-2-1908

Type	Name	Manufacturer or U. S. Supplier
Open-system	Casagrande	Locally fabricated or several suppliers
	Geonor	Soil and Rock Instrumentation, Inc. 377 Elliot St. Newton Upper Falls, Mass. 02164
	Wellpoint	Local suppliers
	Portland District	Locally fabricated
Closed-system	USBR	Plasticrafts, Inc. 2800 North Speer Blvd. Denver, Colo. 80211
	Bishop	Soil Instruments, Ltd. Townsend Lane London NW9, England
Diaphragm		
Pneumatic	Warlam	A. A. Warlam Box 122 Saddle River, N. J. 07458
	Hall	Geo-Testing, Inc. P. O. Box 959 San Rafael, Calif. 94902
	Dames & Moore	Dames & Moore 2333 West 3rd St. Los Angeles, Calif. 90057
	Terra Tec Thorpezio	Terra Tec, Inc. 250 N. E. 49th St. Seattle, Washington 98105
Hydraulic	Terrametrics Hydrostatic Pore Pressure Cell	Terrametrics 16027 West 5th Ave. Golden, Colo. 80401
	Gloetzl	Terrametrics
Electric strain gauge	Carlson	Terrametrics
	WES Transducer	U. S. Army Engineer Waterways Experiment Station P. O. Box 631 Vicksburg, Miss. 39180
	University of Alberta, GSC	Locally fabricated
Electropneumatic	Pore Pressure Transducer	Slope Indicator 3668 Albion Place North Seattle, Wash. 98103
Electrical acoustical	Maihak	Soil and Rock Instrumentation, Inc.
	Telemac	Soil and Rock Instrumentation, Inc.
	Geonor	Soil and Rock Instrumentation, Inc.

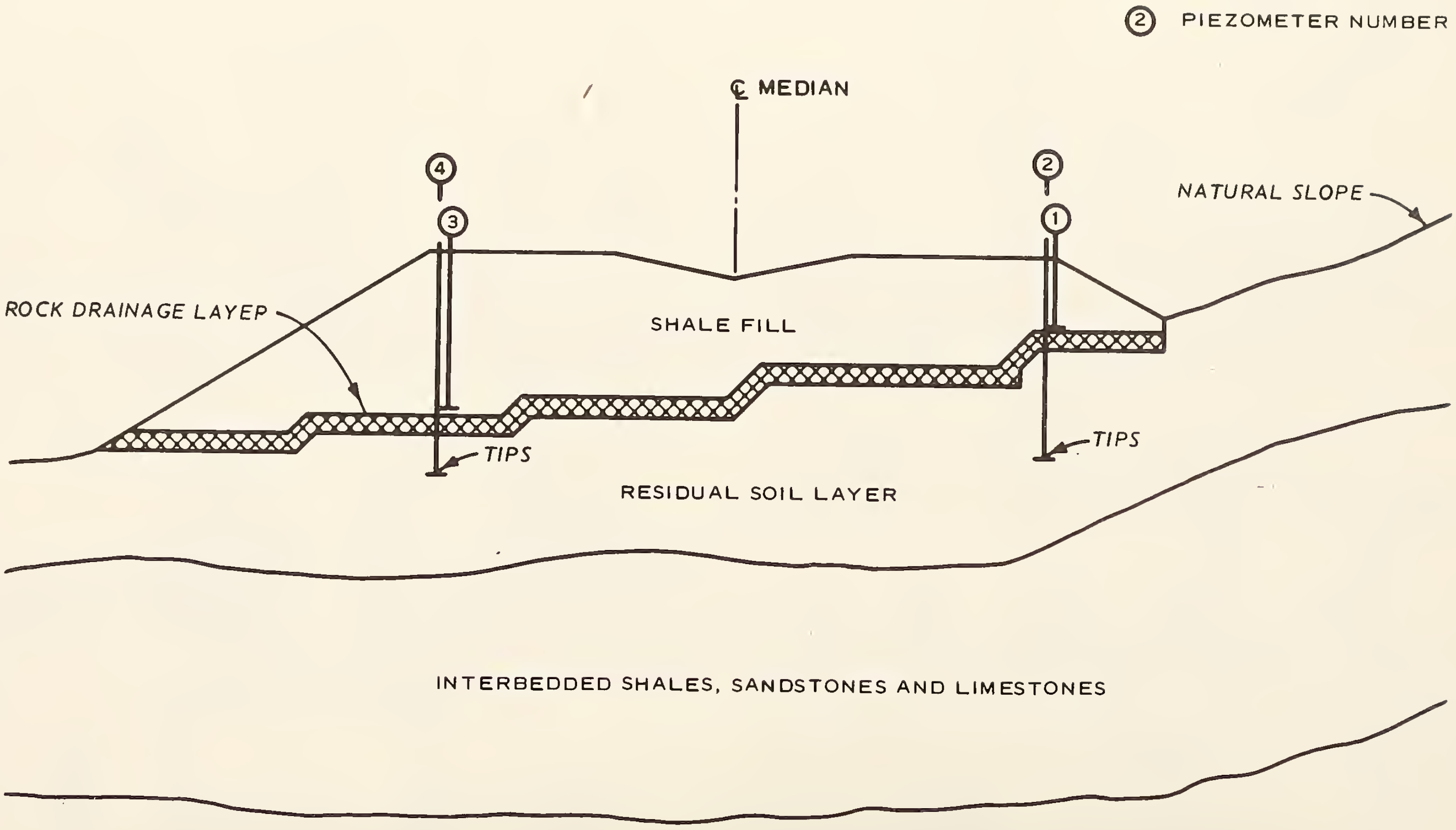


Figure 38. Typical piezometer locations

_d_. Character of fill material.

_e_. Water-level fluctuations within the fill.

_f_. Schedule of reading of other instrumentation.

_g_. Proximity of personnel for making readings.

The evaluator must consider all the factors involved and obtain an optimum interval for reading the piezometers.

Surface-movement measurements

197. Distress may first appear as horizontal and lateral movement of the fill. When this occurs, an indication of the seriousness of the problem and the magnitude of future movements may be obtained by observing the movement. Conventional surveying methods can be employed for monitoring movements on markers, pins, or stakes placed on the embankment surface.

198. Establishing lines of markers for measuring settlement and horizontal movement can be helpful in locating potential failure surfaces. These lines can be established on the roadway, on the embankment slope, and on natural ground at the toe of the slope. Figure 39 gives a recommended positioning of markers for different types of expected movement. Care must be taken in establishing lines for monitoring settlements if horizontal movement is also to be measured. The reference point used in establishing the lines must be outside the zone of expected movement. Because of creep of the slope surface (Figure 39), markers should be installed such that the upper portion of the marker is not in contact with the fill material. This may be accomplished by drilling a 4- to 6-in.-diameter hole to a depth of about 5 ft, casing it, and then driving a reference rod approximately 2 ft below the bottom of the casing (Terzaghi and Peck, 1967). Horizontal and vertical displacements can then be measured using the top of the rod. Figure 40a shows typical lines (A and B) established on a California fill for monitoring settlement and horizontal movement (Durr, 1974*). Figures 40b and 40c show the results of observations.

199. Simple profiling is recommended as the most useful method of obtaining deformation data on roadway embankments. Periodic cross-section surveys are very useful in distinguishing between settlement and bulging of a fill. The time interval between periodic profiles and cross sections depends upon the rate of deformation.

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* Durr, D. L., "An Embankment Saved by Instrumentation," Transportation Research Record 482, Landslide Instrumentation, Transportation Research Board, Washington, D. C., 1974.

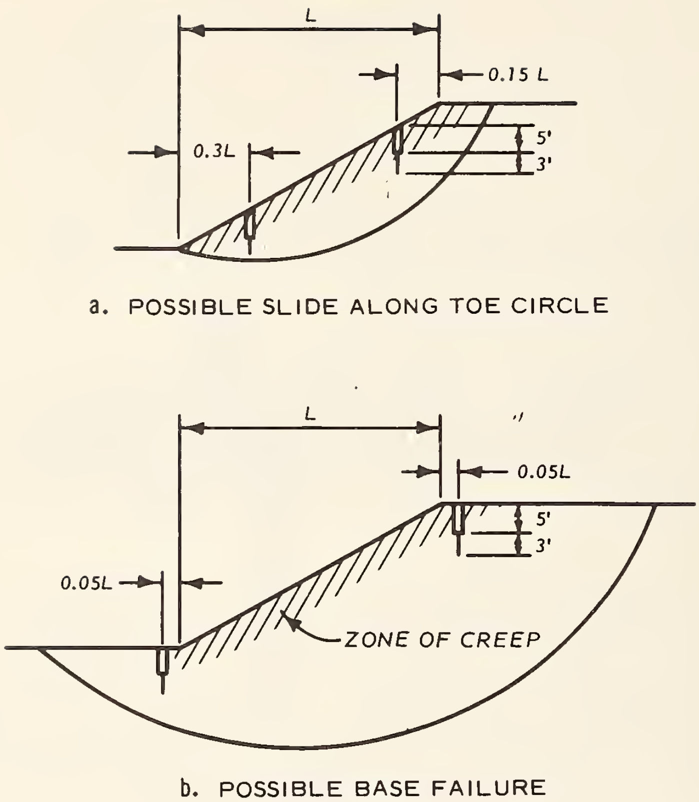

Figure 39. Position of reference points to detect movement of slopes (after Terzaghi and Peck, 1967)

200. Another type of surface measurement is the monitoring of changes in the width of crack openings. The measurements are generally made using referenced markers located on both sides of the crack simply by taping the distance between markers or by using more complicated devices (Vol. 2).

Movement within embankments

201. The monitoring of movement at depths within an embankment is also important in the detection and possibly the prevention of failures. When an embankment is unstable, movement often begins long before failure. The recommended method of obtaining horizontal displacements is by the use of inclinometers, which are devices for measuring the lateral deflection or tilt of boreholes. From a series of inclinometer observations, the type, depth, magnitude, direction, and rate of lateral movement can be determined. Telescoping inclinometer casing can be used to measure settlement using a special probe having a hook (EM 1110-2-1908, Part 2).

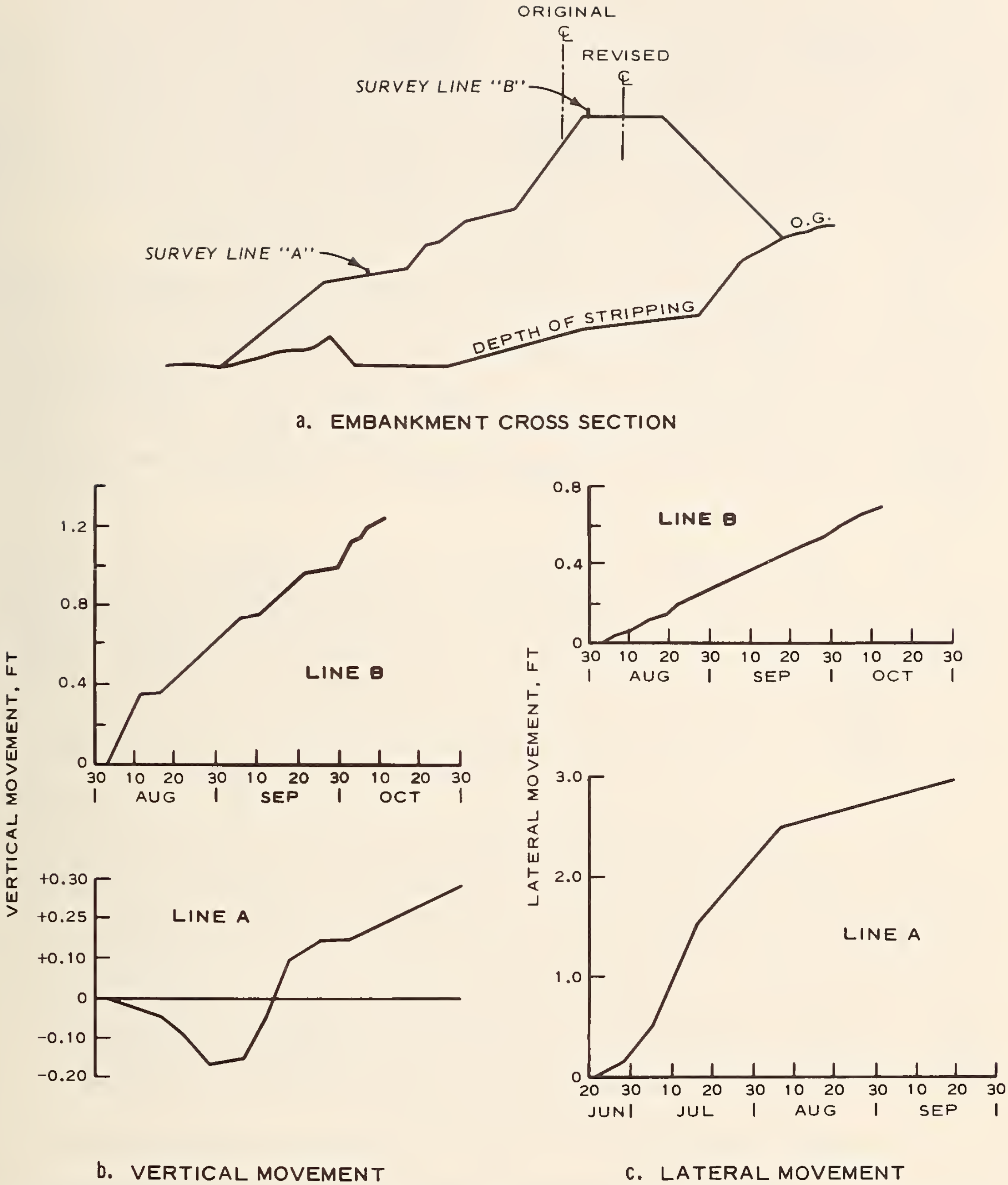

Figure 40. Vertical and lateral movement of highway embankment on U. S. 101 in California (after Durr, 1974)

202. Several types of probe inclinometers are commercially available (Table 17). The apparatus consists of a control box and a probe that is lowered into the casing on a cable. In some probes a cantilevered pendulum with resistance-strain gauges, vibrating wire, or inductive transducers is used to measure cantilever deflection (Franklin and Denton, 1973).* Other probes use the wheatstone bridge principle (Slope Indicator Model 200B), the servo accelerometer principle (Slope Indicator Digitilt), or a differential transformer (Dames and Moore EDR). The probe generally requires a special casing, as shown in Table 17. The electrical output from the probe is measured at the control box and converted to visual display, punched tape, or graphic form.

203. Inclinometer casing should be installed in a nearly vertical hole that intersects the failure zone.** The hole should extend beyond the zone of expected movement and at least 15 ft into soil or rock in which no movement is anticipated. Allowance should be made for loss of the bottom 5 ft of the hole where sediment accumulation occurs. Casings over 50 ft deep should be checked for twist using commercially available equipment, since some of the casings may be received from the manufacturer with a built-in twist that would cause considerable errors in observations (EM 1110-2-1908).

204. When inclinometer casings are placed in embankments having large boulders or rock fragments, it is recommended that the casings be grouted in or carefully backfilled with various combinations of gravel and sand. If grout is to be used as a backfill for inclinometer casings, weak grout mixtures are recommended. Strong grouts are likely to produce premature blockage of the casing. The disadvantage of sand or gravel backfill is that several weeks may be required for the backfill to stabilize. During such time, readings may be misleading.

205. In cases where the embankment distress has not proceeded to a point at which the lateral extent of the failure can be determined or when no indications other than settlement are apparent, inclinometer casings should be located at intervals along the suspect embankment. Installation of inclinometer casings along the slope is desirable but not always practical. A more complete description of the type and extent of movement (such as a failure plane or general bulging) is obtained with each inclinometer casing that penetrates the embankment. Figure 41 illustrates the use of inclinometer installations along a slope to delineate the failure plane for various types of failure.

206. The frequency of observations depends on several factors,

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* Franklin, J. A. and Denton, P. E., "The Monitoring of Rock Slopes," The Quarterly Journal of Engineering Geology, Vol. 6, No. 3-4, 1973.

** Measurements in nonvertical holes can be made with some inclinometers; however, before planning such holes the engineer should be aware of the limitations of the particular inclinometer being used (Table 17).

Table 17. Probe Inclinometers as Given in EM 1110-2-1908 and Franklin and Denton, 1973

Type	Trade Name	Approximate Casing Size mm	Casing Type	Range mm/m	Range deg	Sensitivity mm/m	Sensitivity sec	Manufacturer
Strain-gauged pendulum	CRL Inclinometer	45 x 45	Square aluminum duct	±88 from vertical	±5	0.075	15	Cementation Research
	Inclinometer	50	Aluminum tubing with keyways	360 from vertical	±20	0.2	36	Soil Instruments
	Borehole clinometer	76 x 76	Square steel tube	±175 from vertical	±10	0.1	20	Structural Behavior Eng. Lab.
	C-350 slope meter	45 x 45	Square steel tube	±577 from vertical	±30	0.075	15	Soiltest
Pendulum with rheostat	Series 200-B slope indicator	81	Aluminum tubing	±467 ±87 from vertical	±25 ±5	1.0	180	Slope Indicator
2 electrolevels at 90 deg, servomotor and compass	Slope reader	51	Plastic	±175 from vertical	±10	0.1	20	Eastman
Servo accelerometers	Digitilt	30/70/81	Aluminum/ plastic tube	±577 infinite	±30 ±90	0.1	18	Slope indicator
Pendulum with vibrating wire, 2 direction, compass or keyway	MDS 83	50 or larger	Aluminum or plastic, key-ways optional	±290	±15	0.05	10	Maihak
Pendulum with vibrating wire	68-062 inclinometer	50	Aluminum alloy	±792	±45	0.15	30	ELE/Geonor
Pendulum with differential trans-former, automatic recorder	Earth deformation recorder (EDR)	89	Plastic with grooves			0.3% for angles up to 4 deg; 0.15% for angles up to 8 deg		Dames & Moore
Pendulum with vibrating wire	MPF clinometer				15	1		Telemac

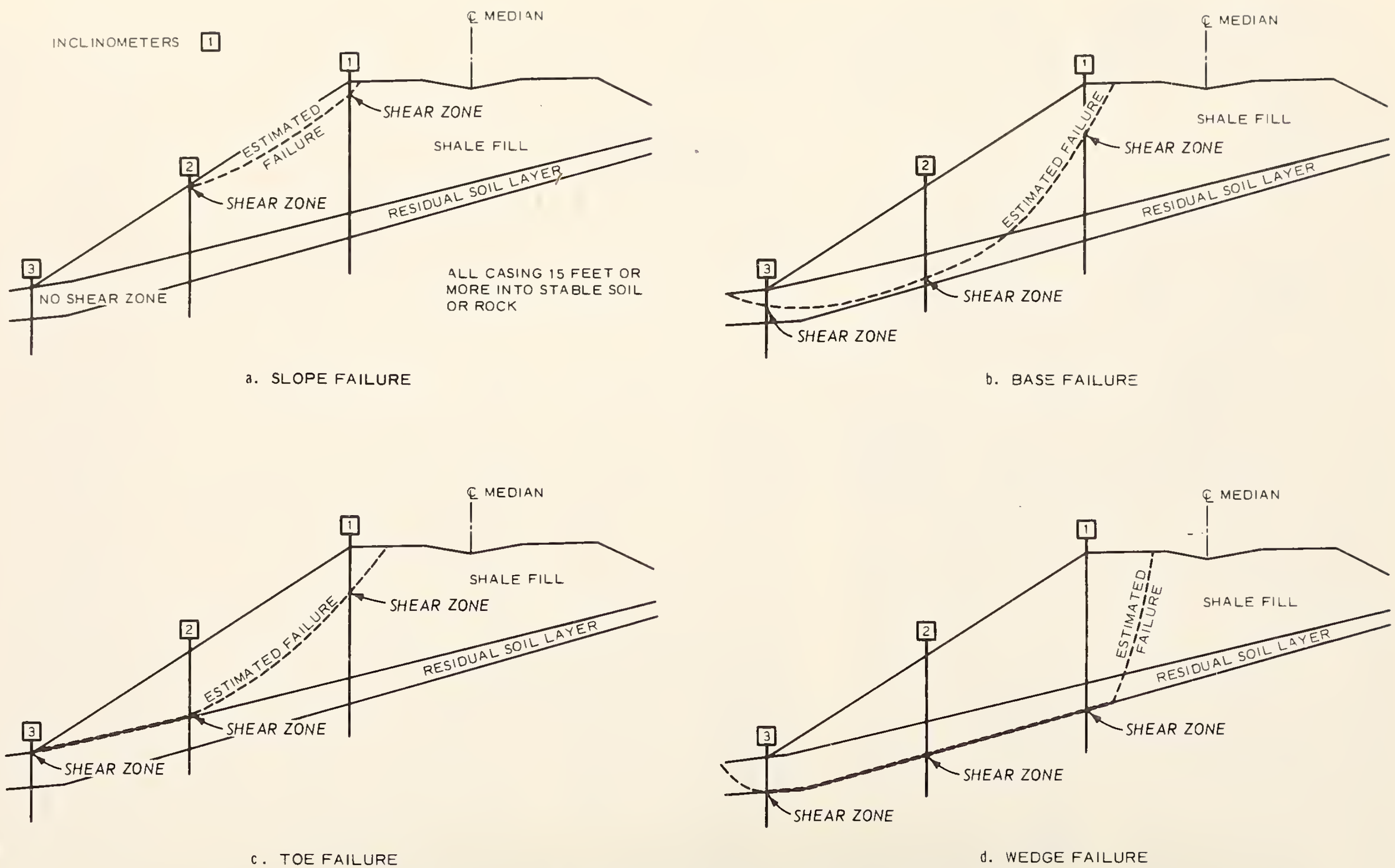

Figure 41. Estimation of failure surface from slope inclinometer data

the most important of which is the rate of movement. It is necessary to read inclinometers frequently after installation, and based on initial observations, to adjust the interval of observations. Observations of piezometers, settlement devices, and other instrumentation should be coordinated closely with inclinometer observations.

207. The primary purpose of gathering inclinometer data is to delineate the zone of movement. This zone of movement can then be used in the analysis of sliding stability of the embankment with failure surfaces estimated as in Figure 41. Movement may also indicate consolidation of the foundation (Figure 42). Movement from consolidation would necessitate further analysis of the compressibility of the foundation to determine the magnitude of the stresses induced in the embankment due to consolidation. Stresses could simply cause displacements, or they could be great enough to exceed the shear strength of the fill.

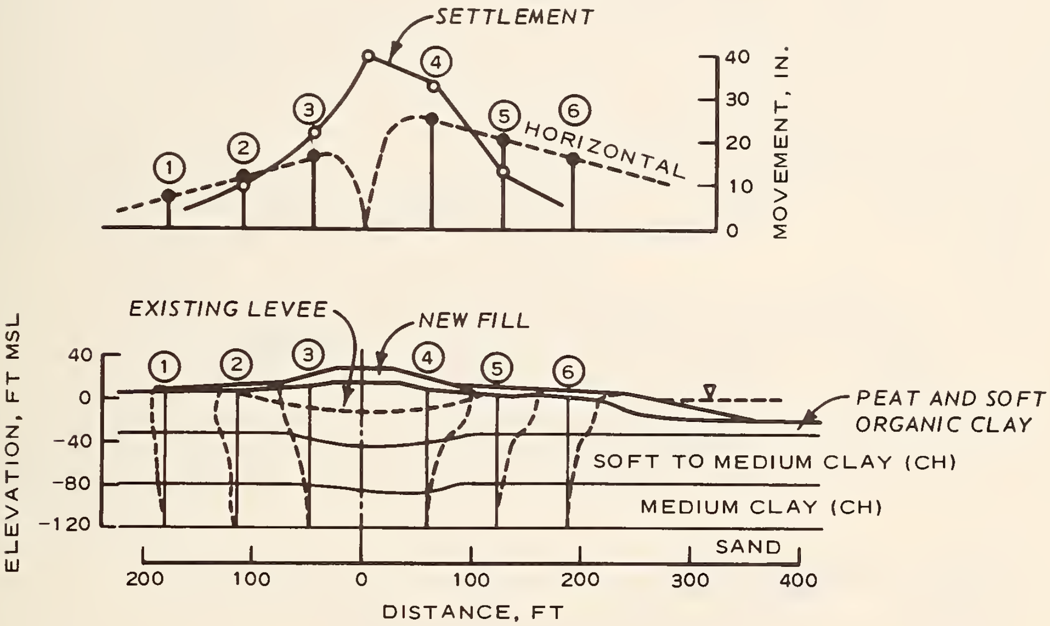

Figure 42. Foundation movement indicated by slope inclinometer data, Atchafalaya Levee, Louisiana (Wilson, 1970)

208. The plotting of movement data in a manner consistent with piezometer, rainfall, and stream-flow data is again recommended. Such a plot might indicate a trend with movements occurring at a greater rate following periods of heavy rainfall or at periods of high stream flows. Several types of plots can be made from inclinometer data. Movement versus time plots are used to show a rate of movement (Figure 43*).

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* Wilson, S. D., "Observation Data on Ground Movements Related to Slope Instability," Journal of the Soil Mechanics and Foundation Division, American Society of Civil Engineers, Vol. 96, No. SM5, Sep 1970.

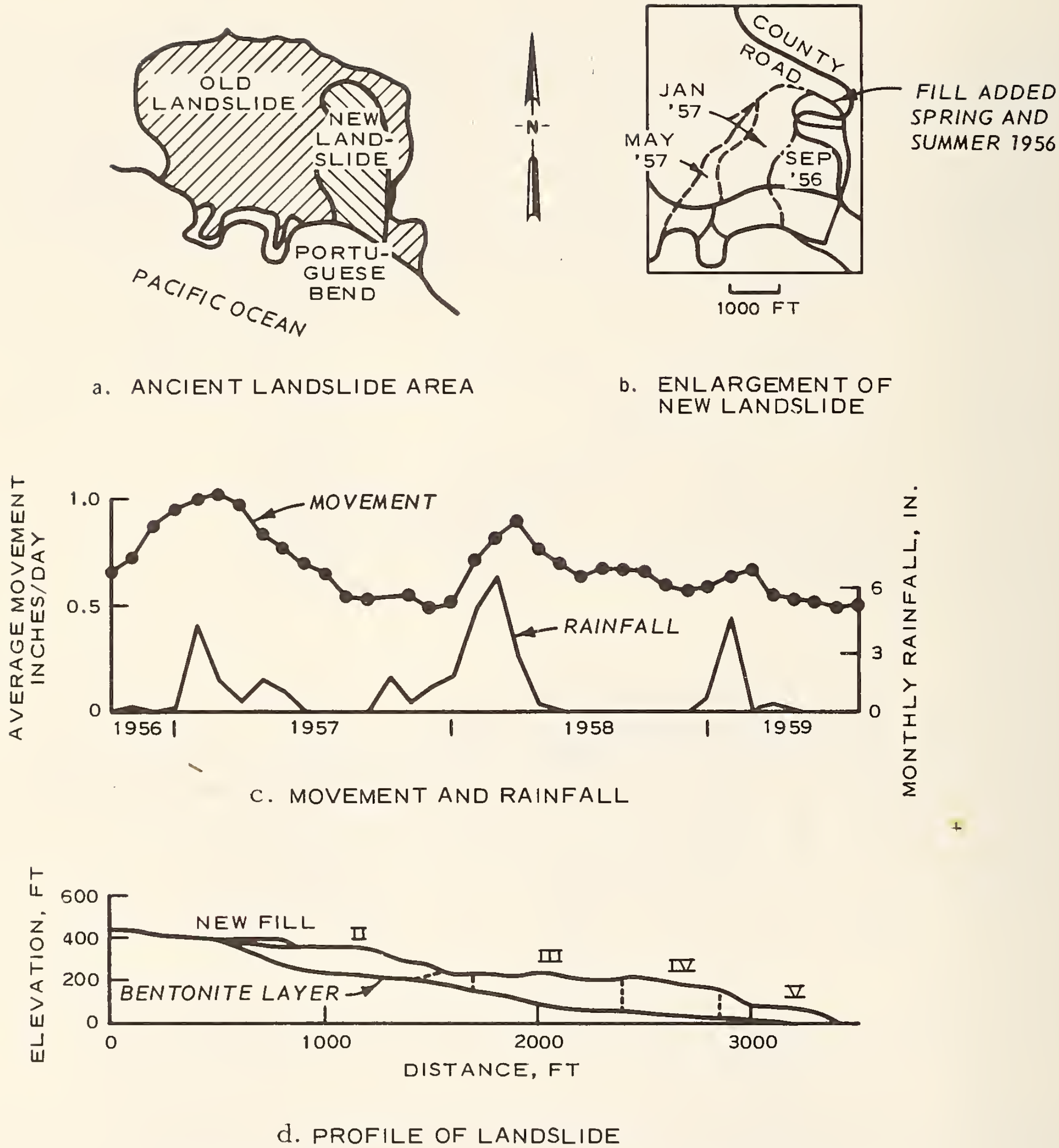

Figure 43. Movements at Portuguese Bend Landslide, California (Wilson, 1970)

It should be noted in this figure that the movement is also accelerated by rainfall. This relationship indicates the importance of the comparison of inclinometer data with other available information such as rainfall records. A plot of movement versus depth enables the determination of the zone in which movement is occurring (Figure 44*).

209. Further guidance concerning the use of inclinometers is contained in EM 1110-2-1908; Leach, 1976;** and manufacturer's literature.

Monitoring seepage and drain discharge

210. Seepage along the toe of the embankment and on the slopes should be monitored periodically and especially after rainy periods. Low altitude remote sensing (Part III, paragraphs 30 to 33) is a rapid method for detecting seepage from shale embankments along a selected section of highway. If possible, seepage discharge should be measured; however, an estimate of the quantity of flow relative to that normally observed along with piezometer observations can provide some indication of embankment permeability. If drains have been installed then, drain discharge measurements can also help determine whether the embankment is draining. These measurements or estimates should be plotted along with piezometer observations and reported rainfall.

Collection of Strength and Compressibility Data

211. The evaluation of embankment stability requires that the strength and compressibility of the embankment materials be determined. The determination can be made by considering information gathered as the result of:

a. Sampling and laboratory testing of fill materials.

b. In situ measurement.

c. Back analysis of failures.

d. Experience with similar problems.

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* Hopkins, T. C., "Settlement of Highway Bridge Approaches and Embankment Foundations, Bluegrass Parkway Bridges Over Chaplin River," Part II, Research Report 356, Kentucky Department of Highways, Division of Research, Lexington, KY, Feb 1973.

** Leach, R. E., "Evaluation of Some Inclinometers, Related Instruments, and Data Reduction Techniques," Miscellaneous Paper No. S-76-12, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS, Jun 1976.

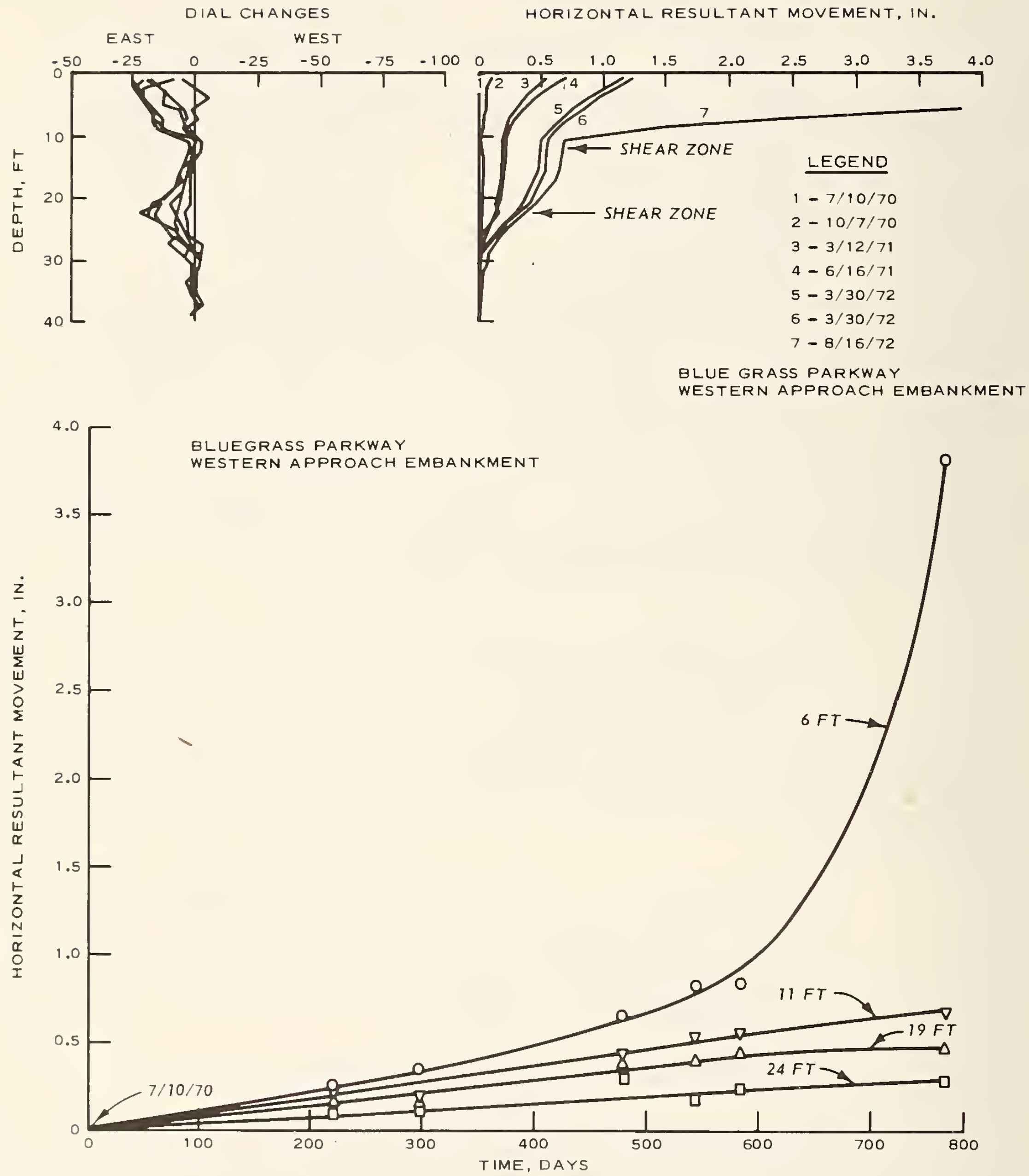

Figure 44. Slope inclinometer results, western approach embankment, Chaplin River Bridges, Bluegrass Parkway, Kentucky (Hopkins, 1973)

The particular method of gathering strength data depends upon the class of the embankment problem and the composition of the fill. Shear strength data for several shale embankments is summarized in Vol. 4. The methods and their application are briefly discussed in the following paragraphs.

Sampling and laboratory testing

212. The fill characteristics largely determine the extent to which sampling and testing will be successful. Investigations have shown that placement of shale in embankments may vary from thick layers of rock pieces to thin layers of well compacted soil (Vol. 1). When large particles of hard shale, sandstone, or limestone are present in a fill, conventional undisturbed tube sampling methods are practically useless. Good core samples (5-in. diameter) using a double-tube core barrel and compressed air can be obtained as described in Vol. 4. Disturbed samples to determine the character of materials can be obtained using a 3-in. outside diameter drive sampler. However, recovery may be low and hard rock chunks may be pushed out of the way and missed by the sampler. In spite of this drawback, samples of moist soillike layers are usually obtained, and this material can be remolded in the laboratory to estimated in situ densities (obtained from a nuclear density log of the hole using portable equipment) and tested for undrained shear strength.

213. Laboratory testing of undisturbed or remolded samples conducted for evaluation studies are primarily the same as those conducted for design studies on compacted samples (paragraph 124). Testing of undisturbed samples can include measurements for determining unit weight, permeability, consolidation, triaxial compression, unconfined compression, and direct shear. Disturbed samples can be used to determine water contents, Atterberg limits, specific gravity, grain size, and compaction characteristics.

214. In assigning laboratory tests for determining shear strengths, an effort should be made to duplicate field conditions as nearly as possible. The consolidated undrained (Q) triaxial test on undisturbed core samples with pore pressure measurements is recommended as the test most nearly duplicating existing conditions of a fill. When warranted for major embankments, K_oCU triaxial compression tests (Vol. 4) should be considered to better duplicate in situ conditions. Determinations of a short-term safety factor should use these test results. Consolidated undrained tests with pore-pressure measurements (if possible) conducted on samples remolded to in situ density and water-content can be used to determine strengths for both total stress and effective stress. For determination of a long-term factor of safety, the direct shear test (or repeated direct shear tests for residual strengths) is recommended.

In situ measurement

215. Several devices are currently available for determining properties of materials in situ. Plate-bearing and large-scale direct shear are expensive, limited in application, and should be used only for major embankments as a last resort. Consequently, these tests are not recommended for wide use in evaluation of shale embankments. *Other devices, such as the pressuremeter and borehole shear device, are relatively inexpensive borehole devices that are recommended for use.*

216. The Menard pressuremeter and British Camkometer are currently being used for testing a variety of materials from cohesive and cohesionless soils to rocks. These devices involve measuring the volume changes and/or deformation accompanying the expansion of a cylindrical probe in a borehole. Some of the devices are self-boring, but these have not been proven in shale fills or rocky fills. Appendix B describes the pressuremeter equipment and test procedures. Pressuremeter data can be used to estimate the horizontal modulus. The data can also be interpreted to obtain reasonable estimates of both ϕ and c or simply the undrained strength, s_u, as described in Vol. 4.

217. The Iowa borehole shear test device provides a field test that is similar to the laboratory direct shear test. A normal force is applied hydraulically to two curved metal surfaces bearing on opposite sides of a borehole. A tangential force is then applied and increased until a shear failure occurs in the material close to the plate. The normal and tangential forces at failure are each divided by the area of the plates in contact with the material to give the normal stress, σ, and the shear stress, τ. To determine shear strength, shear stress versus normal stress is plotted for at least three tests and a straight line is fitted to the data. Cohesion (c) is the intercept at zero normal load and the friction angle (ϕ) is the slope of the line. The Iowa shear device should be considered for use in evaluating soillike embankments. Where possible, undisturbed samples should also be taken.

Analysis of Present Conditions and Prediction of Future Behavior

218. After gathering data on a problem shale embankment from all available sources, the geotechnical engineer must assess the validity of the data and evaluate the future behavior of the embankment. If continuing settlement is the problem, the cause must be determined and an assessment made of the extent, rate, and probable amount of future settlements. Should slope stability be in doubt (e.g., for a sidehill embankment on weathered shale), slope stability analyses should be made to determine the existing factor of safety. At times, a failure will have already started and immediate remedial treatment will be necessary. For these cases, appropriate strength properties must be selected for

use in the design of remedial treatment measures. The following paragraphs describe these evaluations and assessments with reference to the application of data accumulated through instrumentation, sampling and laboratory testing, and in situ investigations.

Settlement

219. Settlement of shale embankments is a major type of distress caused by infiltrating water, resulting in softening and compression of nondurable shale often mixed with soil and/or durable rock (Vol. 4). Because random distribution of materials frequently occurs (e.g., shale and rock, overburden soil, and weathered shale placed in different parts of different layers), infiltrating surface water and subsurface seepage can follow erratic flow paths. The result is nonuniform settlement, which is difficult to evaluate. Estimating how long such settlements will continue, what total settlement will occur, and whether a slide will occur requires considerable experience and judgment based on a thorough evaluation of field monitoring data, boring logs, site geology, and construction records.

220. Settlement data from periodic profile surveys of spikes along the roadway and monuments set in the side slopes can show the settlement distribution, influence of rainfall and future trends (by extrapolation). If surface drainage is ponding in the median and/or wet areas exist on the side slopes, continued settlements may lead to slope failure.

221. Lateral movement data from periodic surveys of slope monuments can indicate slope bulging when correlated with slope indicator data that show a relatively uniform increase in lateral movement with depth in the embankment. Boring logs and samples of in situ materials may indicate a predominance of granular materials, in which case it would be expected that deformations would stabilize in one to several years, depending on in situ densities (see Figure 27) and subsequent rainfall intensity. Poor surface drainage conditions and site geology showing permeable strata dipping into the embankment (Figures 4c and 4b) could prolong the settlements.

Estimating additional settlement

222. With sufficient information and relative uniform embankment materials versus depth, additional settlements can be roughly estimated by two methods: compression tests and modulus values.

223. Compression test. Using the test procedure outlined in Vol. 4, shale materials representing those in the embankment can be remolded (by static compression or kneeding compaction*) to in situ

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* Abeyesekera, R. A., "Stress-Deformation and Strength Characteristics of Compacted Shale," Joint Highway Research Project JHRP-77-24, Purdue University, West Lafayette, IN, Dec 1977.

densities and water content in a 6-in.-diameter greased CBR mold and the compression measured under an applied load. The applied load should simulate the existing overburden load at the midheight of the represented material layer. Materials containing large quantities of plus 3/4-in. rock can be scalped. The measured percent compression for in situ or expected moisture conditions (moist, wet, or saturated) times the represented thickness of embankment can be used to judge the expected additional compression of the embankment, as shown in Figure 45.

224. Modulus value. If representative values of tangent modulus of elasticity can be measured from tests on undisturbed or remolded samples (to in situ conditions) or from pressuremeter tests (assuming the modulus is equal for vertical and horizontal directions), the additional settlement can be roughly estimated using procedures outlined in Vol. 4 (paragraph 77). For major embankments (over 50 ft high) where future settlement and deformation predictions are critical and an adequate investigation can be undertaken, testing and analytical procedures described by Nobari and Duncan (1972) (see paragraph 129) may be appropriate.

Slope stability

225. The determination of factor of safety of shale embankment slopes against sliding can be made using laboratory strength data from tests on undisturbed samples (or reconstituted samples) or strength data from in situ tests. Guidance on stability evaluations are given in paragraphs 130 to 134 and in Vol. 2. A comprehensive investigation by D'Appolonia Consulting Engineers Inc.* for the Indiana State Highway Commission is recommended as a case study reference.

226. When a slope failure has occurred or a shear plane has been identified, in situ strengths can be estimated by using all available data to reconstruct the slope geometry and incipient sliding path and performing a back analysis of the slope. A back analysis makes use of the fact that the factor of safety (FS) at failure equals 1.0. To perform a back analysis, it is necessary to:

- a. Describe the existing slope geometry and failure path (plane, circular, wedge, or combination) just prior to failure.
- b. Describe the internal pore-water pressures and external loads acting on the embankment at failure.

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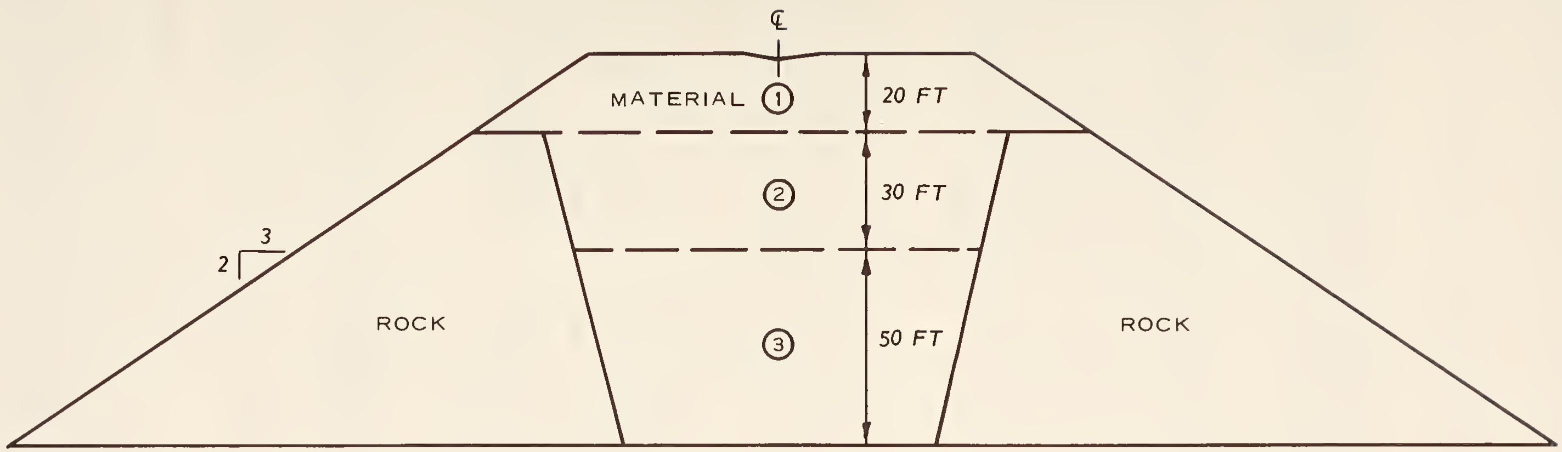
* D'Appolonia Consulting Engineers, Inc., "Summary of Investigation and Recommendations, Evaluation of Embankment Stability (ISHC Project No. I-74-4(73)(63))," Final Report in Four Volumes, Indiana State Highway Commission, Indianapolis, IN, Nov 1977.



MATERIAL TYPE	PERCENT COMPRESSION	PROPORTIONAL HEIGHT, FT	ADDITIONAL SETTLEMENT, FT
①	0.5	20	0.1
②	1.5	30	0.4
③	1.0	50	0.5
		TOTAL	1.0 FT

Figure 45. Example of using percent compression to estimate expected additional settlement

c. Set the expression for factor of safety for the described conditions equal to 1.0.

d. Solve for the family of strength parameters that will satisfy the expression.

Since there are an infinite number of combinations of ϕ and c that will satisfy the back analysis, an assumption is sometimes made regarding one of the variables. Typical assumptions are that either ϕ or c is equal to zero, depending on the soil type and pore-water pressures.

227. The failure surface can be described when data from slope inclinometers are available, as previously discussed and shown in Figure 41. Field measurements can also locate the failure surface at the crest of the embankment and the area of bulging at the toe. When subsurface data from inclinometers, shear strips, or other devices are not available, the shape of the failure surface between the crest and the toe can be estimated by considering boring logs and locations of soft or weak zones or layers. Piezometric data, if available, can be used to estimate pore pressures. Otherwise, an assumption of the pore pressures at failure must be made.

228. The solution of the expression for factor of safety using slope-stability computer programs usually involves successive trials, since most programs search for the critical failure surface for given values of ϕ and c.* Each trial requires changing ϕ and/or c until a factor of safety near 1.0 is obtained for a failure surface conforming to the actual failure surface. When this is done, one particular set of values for ϕ and c is defined that could exist at equilibrium (FS = 1.0) for the failure surface and pore pressures analyzed. An alternate method involves an analysis using a particular ϕ and c for the failure surface, division of $\tan\phi$ and c for the failure surface by the resulting safety factor, plotting c/FS versus $(\tan\phi)/FS$, and repeating for different values of ϕ and c. The individual points on the plot (Figure 46) will fall in a straight line, provided the definition of factor of safety is expressed as:

$$FS = \frac{\text{available shear strength}}{\text{shear stress required for equilibrium}} = \frac{c + \sigma \tan\phi}{\tau}$$

The resulting plot defines all possible combinations of ϕ and c on the failure plane at failure.

229. The values obtained from a back analysis can be compared with

* The Morgenstern-Price method has provisions for determining the factor of safety for a given failure plane and different values of ϕ and c.

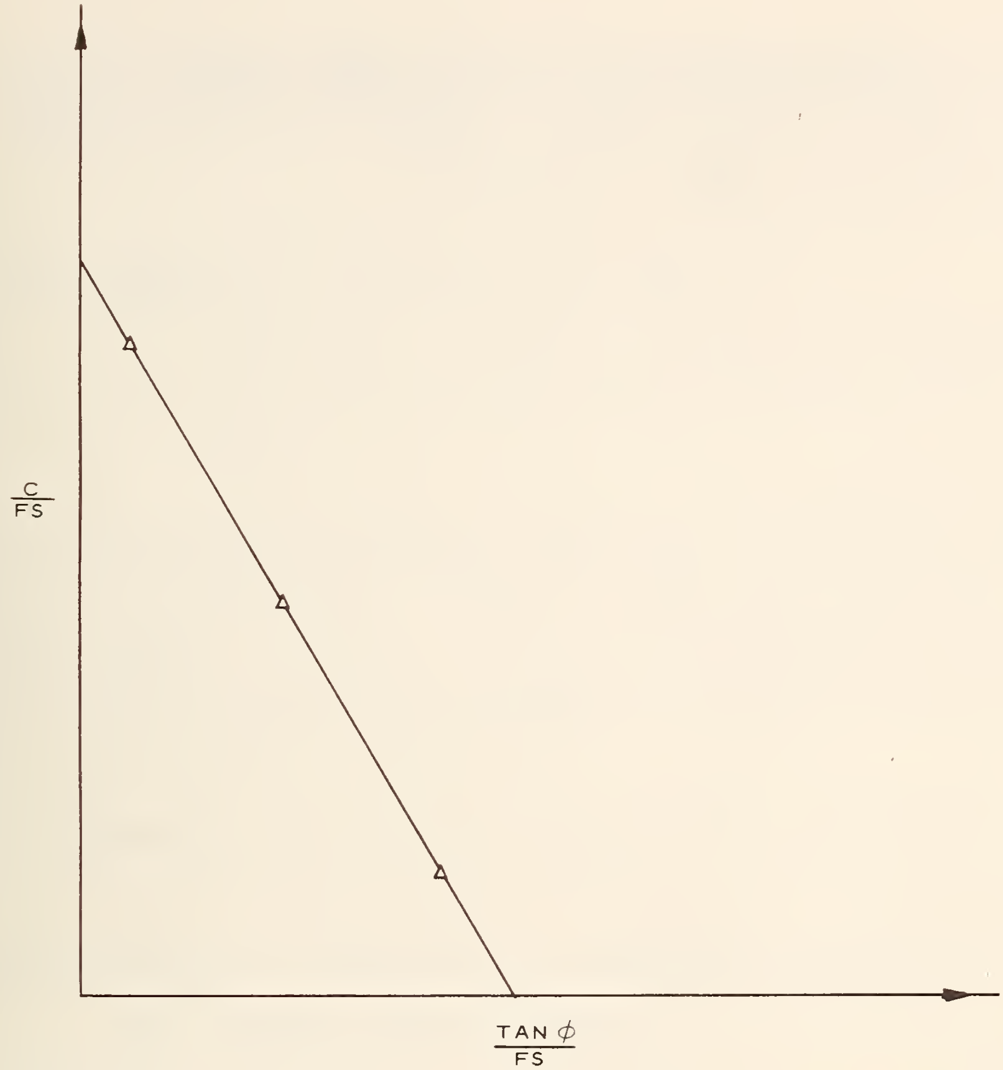


Figure 46. Plot of c/FS versus $\tan \phi/FS$ for description of ϕ and c obtained from back analysis

laboratory data and in situ test data to aid in the selection of strength properties needed for the design of remedial measures. By additional analyses, the effect of fluctuation of the water table upon the strength required for stability can also be determined. Back analyses, correctly performed and interpreted, can provide valuable information for the design of remedial measures. Use of the back analysis technique is described briefly in Vol. 1 (pages 104 to 106).

Experience with similar problems

230. When time constraints, field conditions, or economic considerations do not allow sampling and testing, in situ measurements, or back analysis, the engineer must rely on his judgment and experience with similar embankments constructed from shale materials. This experience will vary widely from engineer to engineer. It is, therefore,

recommended that results of evaluation investigations and studies of shale embankments be documented to provide future references for cases where strengths cannot be measured or determined by one of the three methods mentioned above. A number of reports of this type have been identified in Vol. 1 and 2.

PART IX: REMEDIAL TREATMENT OF SHALE EMBANKMENTS

Introduction

231. Discussion and recommendations concerning remedial treatment of shale embankments are presented in two parts:

   a. Remedial Treatment Methods and Recommended Applications.

   b. Development and Implementation of Remedial Treatment Plans.

In part a are presented remedial treatment methods recommended for permanent and/or temporary treatment of distressed or failed shale embankments. Discussion includes the purpose, design, construction, and recommended applications of individual or combinations of remedial treatment methods. In part b are presented recommended procedures and considerations for development, selection, and implementation of effective and economical site specific remedial treatment plans using primarily remedial treatment methods presented in part a.

Remedial Treatment Methods and Recommended Applications

232. Remedial treatment methods recommended for permanent and/or temporary treatment of distressed or failed shale embankments are:

   a. Pavement overlay (temporary).

   b. Cement grouting (permanent).

   c. Drainage methods (temporary and permanent).

   d. Slope-flattening, berms, shear trenches (permanent).

   e. Retaining walls (permanent).

   f. Embankment reconstruction (permanent).

Remedial treatment methods are discussed below under separate headings. Discussion includes the purpose, design, construction, and recommended applications of individual or combinations of remedial treatment methods. Important considerations relating to treatment of shale embankments are emphasized. Key references are cited, as necessary, to supplement discussions of remedial treatment methods.

Pavement overlay

233. Asphalt overlaying of cracked or settled pavement sections is

a common highway maintenance practice. However, pavement overlaying fails to treat subsurface conditions that can cause pavement and embankment distress. Experience indicates that extended periods (sometimes as much as 5 to 10 years) of pavement cracking and settlement are normally accompanied by a gradual loss in fill shear strength, which generally leads to slope failure in shale embankments. A one-time pavement overlay to correct for settlement caused by minor consolidation within a shale embankment is acceptable. However, periodic pavement overlay at a location without also treating the problem source is poor practice. It is recommended that pavement overlay be accompanied by more permanent remedial treatment methods to minimize the probability of a future slope failure.

Cement grouting

234. Settlement problems in shale embankments are sometimes associated with high porosity fills resulting from inadequate compaction during initial embankment construction and/or possibly long-term internal erosion of soft shales surrounded by more durable shale or limestone particles. Experience summarized in Vol. 2 (page 173) indicates that cement grouting of embankment voids can effectively halt embankment settlement. However, uniform grout penetration is required for effective treatment. For example, in one case where cement grouting did not arrest embankment settlement, a test pit inspection revealed only a few isolated grout masses. Shale materials within this embankment were broken down and sufficiently compacted to prevent uniform grout penetration.

235. It is recommended that cement grouting be conducted when embankment settlements have been attributed to a high percentage of interconnected voids (porosity of 20 to 40 percent). Cement grouting should be applied soon after settlement and associated high embankment porosity are identified. Early application can prevent large-scale pavement deterioration and weakening of the fill. After grouting, fills that are subjected to underground seepage, such as sidehill and transitional fills (see Figures 4 and 21), subsurface drainage measures (for example, horizontal drains) should be installed as necessary to compensate for the sealing effect of cement grout. Engineers experienced in grouting should be consulted to assist in design and implementation of grouting programs. Guidance concerning grouting methods, equipment, and post-grouting field verification is contained in the FHWA manual "Grouting in Soils, Vol. 2, Design and Operations Manual," Report No. FHWA-RD-76-27 (Herndon and Lenahan, 1976) and the Department of the Army Technical Manual, "Grouting Methods and Equipment," TM 5-818-6 (Departments of the Army and Air Force, 1970).

236. In addition to cement grouting of voids, other stabilization methods potentially applicable to shale embankments (Vol. 2) are:

- a. Cement or chemical grouting to penetrate along a well defined shear surface.

b. Chemical grouting of voids.

c. Lime stabilization (drill-hole lime or lime-slurry injection).

d. Electrokinetic stabilization.

e. Ion exchange.

There has been no experience with these stabilization methods in treating shale embankments. These methods are not recommended for routine application. Expert guidance is normally required when applying these techniques, and the effectiveness of these methods is difficult to predict. It is recommended that these stabilization methods be used only on a trial basis at selected sites where risk of failure is minimal and a substantial savings over more conventional remedial treatment methods can be realized.

Drainage methods

237. Surface infiltration and underground seepage have been the major causes of distress and failure in shale embankments (paragraph 18). Consequently, drainage systems will be an integral part of most remedial treatment methods. Early implementation of certain drainage methods is recommended to halt embankment distress before extensive failure can develop. Drainage methods suitable for treatment of shale fills are:

a. Surface treatment.

b. Horizontal drains.

c. Vertical drains.

d. Pumped vertical wells.

e. Interceptor trench drains.

f. Drainage blankets.

238. Surface treatment (a above) includes surface drains and other surface repair or modifications used to control runoff and minimize infiltration of surface water. Surface treatment is an important consideration in remedial treatment of any distressed or failed shale embankment. The remaining drainage methods listed above (b to f) are types of subsurface drains used to collect and control underground seepage. Subsurface drains are essential in remedial treatment of sidehill and transitional fills, since these types of fills are commonly subject to underground seepage from the adjacent natural ground (Figures 4 and 21). The basic design considerations for individual drains and drainage systems are presented below, followed by discussions of individual drainage

methods and their application to shale embankments. Application of subsurface drains in remedial treatment of sidehill fills is emphasized, since sidehill fills have been involved in the majority of past shale embankment stability problems.

239. Design considerations. In remedial treatment applications, drainage systems should be designed using established theoretical and empirical methods combined with engineering judgment and experience. A conservative design of remedial treatment drainage systems is recommended because of inherent uncertainties in embankment and foundation material properties, groundwater conditions, drain construction and reliability, and simplification of assumptions necessary in design. Expert assistance should be obtained and trial and error solutions should be used when necessary. Theoretical and empirical methods for design and analysis of drainage systems is contained in the textbooks, Seepage, Drainage, and Flow Nets (Cedergren, 2d ed., 1977) and Drainage for Highway and Airfield Pavements (Cedergren, 1974). Excellent guidance for applying drainage systems to highway embankments is also contained in the Highway Research Board Synthesis of Highway Practice Report No. 8, "Construction of Embankments" (Transportation Research Board, 1971). Other key references are cited in subsequent discussions of individual drainage methods.

240. Thorough site evaluation (Part VIII) is essential in the design of remedial treatment drainage systems. The extent and reliability of site evaluation will largely determine the accuracy and confidence imparted to the design of remedial treatment drainage systems. A visual inspection of surface conditions and estimates of runoff to be collected and controlled will largely determine required surface treatment. However, design of subsurface drains is more complex and requires investigation of embankment and foundation material properties and groundwater conditions.

241. Design of subsurface drainage systems is dependent on groundwater and pore water pressure conditions; extent of embankment distress and failure; and the types, strengths, and permeabilities of embankment and foundation materials. Subsurface drainage systems should be designed to collect and/or control groundwater seepage to prevent development of excessive saturation levels and pore water pressures within the embankment and foundation. Seepage sources should be determined and estimates should be made of maximum saturation levels and pore water pressures likely to occur during periods of maximum groundwater recharge. Drain types, locations, and capacities should be selected to limit saturation levels and pore water pressures to safe values, as determined from theoretical stability analyses (paragraphs 130 to 134 and 226) and when possible from in situ measurements of groundwater levels and pore water pressures taken during periods of embankment distress (paragraphs 190 to 195).

242. Most of the subsurface drains discussed in this section

consist of pervious drainage materials placed in a blanket, trench, or borehole to collect underground seepage. Detailed characteristics of drainage materials is an important (but sometimes overlooked) consideration in design. To provide maximum drainage capacity, it is recommended that clean, coarse aggregate be used in drain construction. However, coarse drainage materials should not be placed directly adjacent to erodible foundation material or shale fill.* A filter aggregate layer or plastic filter fabric should be placed between the coarse drainage aggregate and erodible materials. The filter aggregate layer (or fabric) will maintain long-term drain operation by preventing infiltration and clogging of the coarse drainage aggregate with soil fines eroded from the material being drained. Slotted or perforated pipes are normally embedded within drainage materials to carry collected seepage water to drain outlets. Slots and perforations must be sized properly to prevent infiltration of adjacent materials. Procedures and criteria for selecting drainage materials and collector pipes are discussed by Cedergren (1976, 1974). Locally available sand and gravel deposits or nondegradable rockfill obtainable from nearby rock cuts should be considered for use as drainage materials. However, processing and testing should be conducted as necessary to ensure that discharge capacity and filter requirements can be met.

243. The choice between using filter aggregate layers or filter fabric is based primarily on construction feasibility, convenience, and costs. For example, the plastic filter fabric is likely to be most convenient for placement along steeply sloping ground. Plastic filter fabric can also be used to wrap slotted or perforated collector pipes in trench drains and between drainage pads or blankets and finer grained erodible shales and soil. General discussion of filter fabrics and their application is given in the March and May 1976 issues of <u>Civil Engineer</u> (American Society of Civil Engineers) and previous references (paragraph 107).

244. The effectiveness of all drainage installations (whether combined with other remedial treatment measures or not) should be checked by postconstruction monitoring of embankment deformations, water discharge rates, groundwater levels, and pore water pressures. Piezometer types and locations should be considered in drainage system design (paragraphs 190 to 195). When possible, observed and predicted groundwater conditions should be compared. Additional drainage measures and/or other remedial treatment should be applied when (a) embankment stabilization does not occur and/or (b) additional lowering of the groundwater level and pore water pressures is necessary to obtain a desired safety factor computed from theoretical stability analyses (paragraphs 130 to 134 and 226).

245. <u>Surface treatment.</u> Poor surface drainage conditions should

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* Site investigations have shown that shale fills normally contain significant quantities of fine-grained soils possibly generated during initial embankment construction or during long-term shale degradation.

be corrected to minimize infiltration of surface water into the shale embankment. Median and side ditches should be paved and existing paved ditches inspected and repaired as necessary. In critical areas, curbing should be installed to direct pavement runoff away from embankment slopes. For example, curbing will likely be needed along superelevated sections where surface water from both lanes drains toward an embankment slope. Depressions within the median or on the embankment slopes should be filled (or graded) to prevent ponding of water. Cracks in the highway pavement, shoulders, median, and embankment slopes should be sealed. An overall surface sealing treatment may be necessary when infiltration is widespread across the roadway (Figure 30) and/or embankment slopes. Types of surface sealing treatments (or impervious membranes) available are reviewed by Snethen (1979).* An impervious surface will also hold moisture in the embankment. Consequently, slopes of embankments subject to underground seepage, such as sidehill and transitional fills (Figures 4 and 21), should not be thoroughly sealed without also providing subsurface drainage.

246. An effective method of preventing surface infiltration from seeping through the embankment is to collect surface infiltration in pavement subdrains. Well designed pavement drainage blankets and shallow side trenches can collect infiltration through pavement cracks, construction joints, medians, and shoulders. Pavement subdrains should be installed, as needed, during initial highway construction. In remedial treatment, pavement subdrains are likely to be economically justified only when large sections of pavement require replacement. Guidelines for pavement subdrain design and installation are contained in the FHWA research and development report, "Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Sections," FHWA-RD-72-30, and the FHWA report, "Implementation Package for a Drainage Blanket in Highway Pavement Systems" (FHWA, 1972).

247. Horizontal drains. A horizontal drain consists of slotted polyvinylchloride (PVC) pipe** (usually 1-1/2 or 2 in. in diameter) inserted into a borehole inclined at a 3 to 20 percent grade to allow gravity drainage. Horizontal drains used in combination with a rock buttress to repair an embankment slide along I-75 in Tennessee are shown in Figure 47. The horizontal drains were installed through the lower portion and toe of the fill. Drains within each row were spaced 10 to 20 ft apart. Some of the drains were nearly 600 ft in length to provide deep penetration into the sidehill foundation. Discharge from the horizontal drains was approximately 400 gal/hour.

248. During drilling, the borehole is continually cased by adding

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* Snethen, D. R., "Technical Guidelines for Expansive Soils in Highway Subgrades," Report No. FHWA-RD-79-51, Federal Highway Administration, Offices of Research and Development, Washington, D. C., 1979.

** Perforated metal pipe is also used.

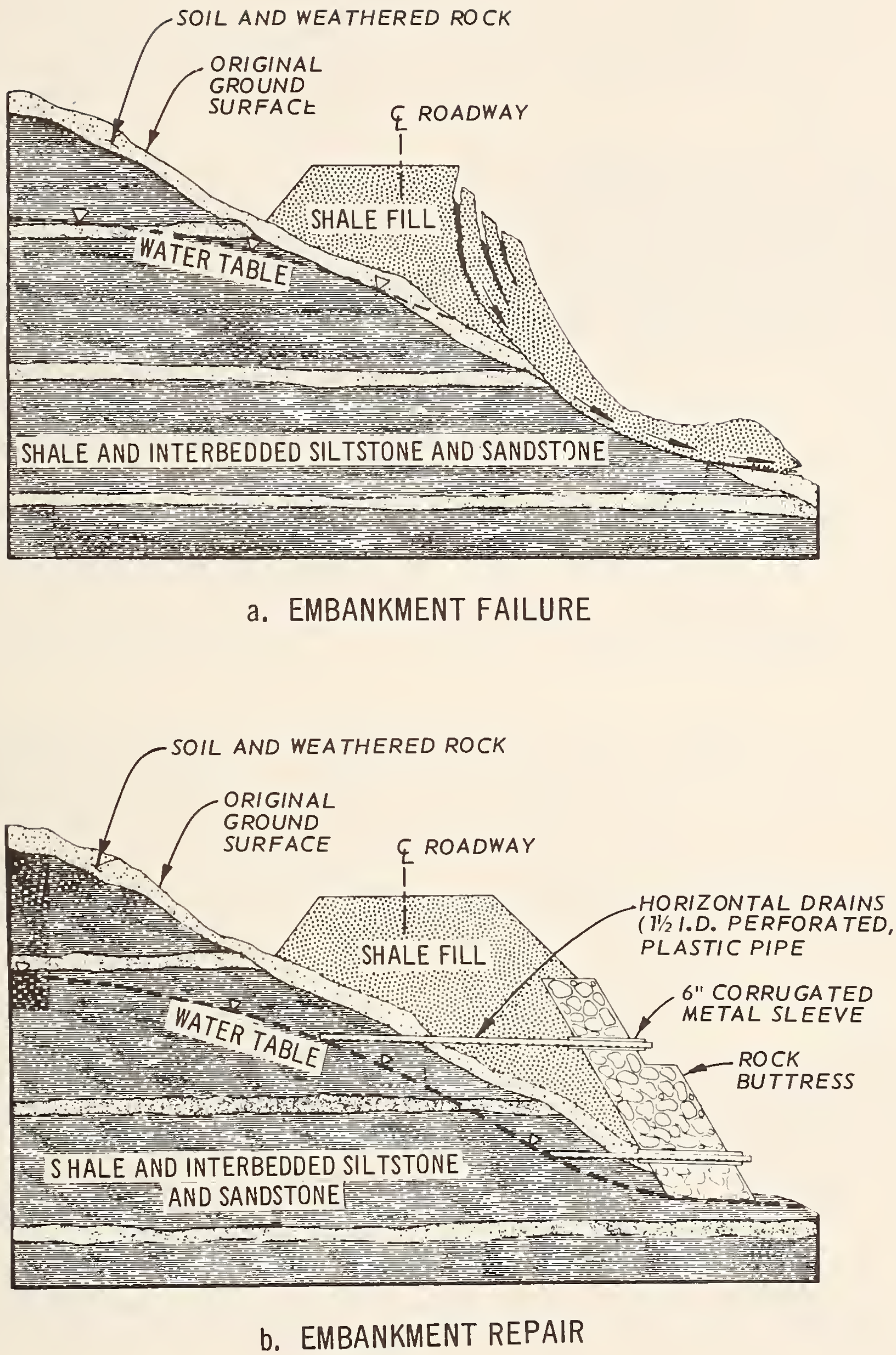

Figure 47. Failure and remedial treatment of shale embankment at sta 1464+00, I-75, Tennessee (Royster, 1973)

sections of hollow steel drill pipe. After reaching the desired penetration distance, the drill bit is detached within the hole and the PVC pipe inserted within the hollow drill pipe. The drill pipe is then removed while the PVC pipe is held in place. This procedure effectively eliminates the possibility of borehole caving prior to the insertion of the PVC pipe.

249. Horizontal drains are susceptible to blockage caused by infiltration of fines or penetration of vegetation roots. The pipe slot width should be chosen to minimize infiltration of fines. If necessary, added filter protection for portions of drainpipe within shale fills or within foundation soils should be obtained by wrapping the slotted PVC pipes with filter fabric prior to their installation (paragraph 242). To minimize blockage caused by root growth, unslotted PVC pipe should be used within 10 to 20 ft of the drain exit. Horizontal drains should be inspected once a year and cleared as necessary to maintain free drainage. Drains can be cleared by water jetting and/or a penetrating drill bit small enough to rotate freely inside the drain. Water intercepted by horizontal drains should be discharged away from the distressed fill. If necessary, paved ditches or 8- to 12-in.-diameter collector pipes should be used to carry off water. Where freezing temperatures are prevalent over extended periods, collector systems should be buried to prevent ice blockage at drain outlets.

250. Horizontal drains are recommended for early treatment of sidehill or transitional fills that are subject to subsurface seepage. Horizontal drains are also recommended for draining water accumulated (through surface infiltration) within porous zones such as limestone or sandstone layers formed during embankment construction (see Figure 4). Early installation of horizontal drains is recommended to help prevent significant deterioration and deformation of distressed shale embankments.

251. In treatment of sidehill fills, horizontal drains should be initiated in the lower portion of the embankment slope or in the natural ground below the embankment toe, as shown in Figure 48. Horizontal drains should be placed in rows spanning the embankment; however, if setup of the drill equipment is a problem, the drains can be arranged in a fan pattern from one or more locations. For example, in repair of a shale embankment slide on I-74, Indiana, 10 horizontal drains were used to supplement a berm and underlaying drainage blanket as shown in Figure 49. The drains (from 124 to 223 ft in length) were separated into three groups, each group arranged in a fan pattern and draining into a separate manhole. The total discharge from the horizontal drains was approximately 900 gal/hour.

252. Before installing horizontal drains, subsurface exploration borings and piezometers should be used to determine embankment and foundation characteristics, groundwater flow pattern, and pore-water pressure distribution. Sidehill foundations commonly consist of

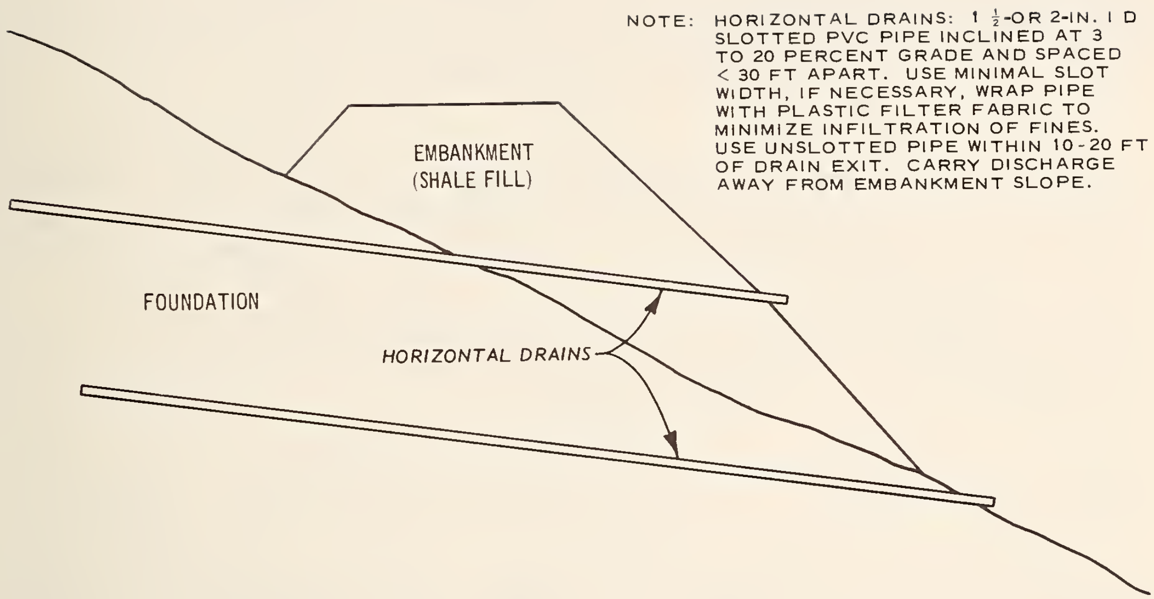

Figure 48. Application of horizontal drains to stabilize sidehill fills

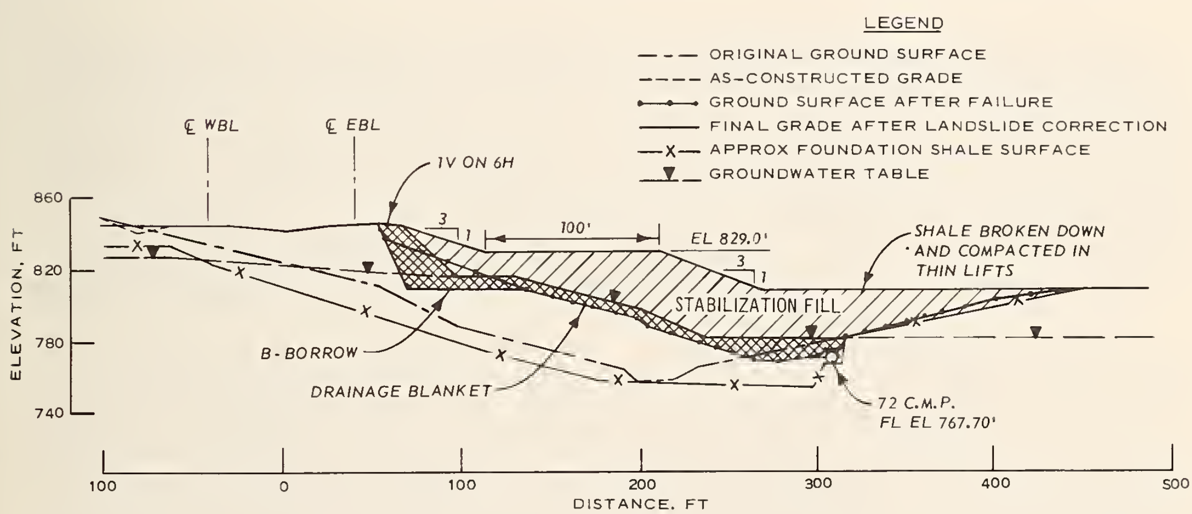

Figure 49. Remedial treatment of a shale embankment slide along EBL, I-74, Indiana (courtesy of the Indiana State Highway Commission)

interbedded shale, limestone, siltstone, and sandstone. Effectiveness of horizontal drains can be improved by intercepting highly permeable foundation layers, such as jointed limestone and sandstone. Horizontal drains should intercept these permeable water bearing layers; however, when these layers are thin and/or widely spaced, borehole deviation during drilling will make it difficult to follow a particular layer. A dense pattern of horizontal drains would be necessary to provide effective drainage. Vertical drains intercepted by horizontal drains should be considered where water bearing foundation layers are thin and/or widely spaced (see discussion, paragraphs 255 to 259).

253. In general, two or more rows of closely spaced horizontal drains will be needed for effective drainage of sidehill fills. Drain spacings within rows should be 30 ft or less, with the closest spacings used at the lower elevations such as immediately below the tow of the fill. Some drains may be located entirely within the fill to drain trapped water; however, most drains should penetrate deep within the adjacent natural ground to intercept seepage before it reaches the fill. Horizontal drains are often too short to be effective. Lengths up to 500 or 700 ft may be required in strata with low vertical permeability. Portions of horizontal drains located within an unstable fill are susceptible to damage from continued embankment deformations. Damage drains within the fill could increase instability by allowing seepage to penetrate, but not pass through the fill. Where fill deformation continues, horizontal drains penetrating the fill should be checked frequently for damage. If drains are damaged and blocked within the fill, additional drains should be installed and/or other remedial treatment should be applied.

254. After installation, the discharge rate from individual horizontal drains should be measured periodically. Even in a successful installation, approximately 30 percent of the drains can be expected to yield little or no water. Changes in groundwater levels, pore-water pressures, and embankment deformation should be monitored. Additional horizontal drains should be installed and/or other remedial treatment applied when (a) embankment stabilization does not occur, or (b) an additional lowering of the groundwater level and pore-water pressures is necessary to obtain a desired FS computed from the stability analysis.

255. Vertical drains. A row of closely spaced, interconnected, vertical drains placed in the natural slope behind sidehill fills or in the natural slope adjacent to transitional fills should be considered where thin water bearing foundation layers (e.g., limestone or sandstone seams) are separated by relatively impervious material (e.g., massive shales). Vertical drains are advantageous because underground seepage will be intercepted before reaching the shale embankment. Vertical drains should be intercepted by horizontal drains to provide gravity drainage, as shown in Figure 50. Vertical drains are recommended for early treatment to help prevent significant deterioration and deformation of distressed shale embankments. Because of the additional expense

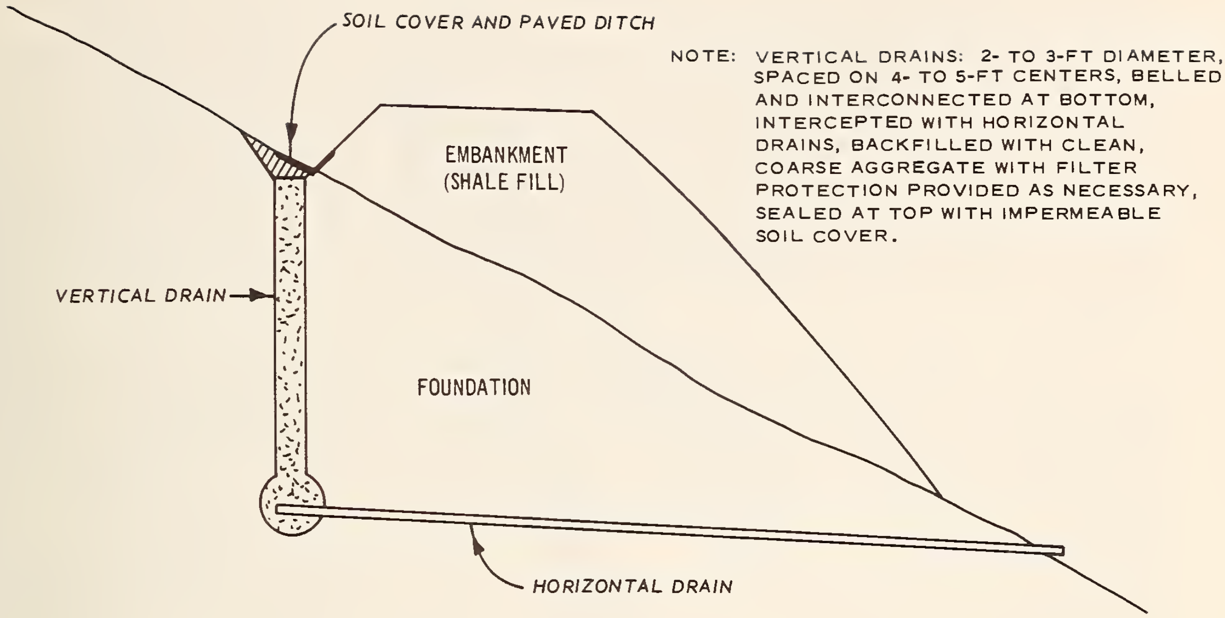

Figure 50. Application of vertical drains to stabilize sidehill fills

of vertical drains, they should be used only when (a) exploration borings indicate that water bearing layers are numerous, thin, and separated by impervious strata, and (b) horizontal drains are found to be ineffective.

256. In treatment of sidehill fills, vertical drains should be placed in the natural slope behind the fill in a single row extending the length of the embankment. Drains should be completed individually or in pairs to minimize the risk of borehole caving prior to backfilling. Vertical drains should be 2 to 3 ft in diameter and spaced on 4- to 5-ft centers. A continuous drainage gallery should be formed by interconnecting the drains. Drains are interconnected by belling out the bottom of each borehole. Before backfilling, the borehole walls should be washed to remove any mud that may have formed during drilling. Also, a perforated pipe should be placed in some of the boreholes to allow later monitoring of the groundwater level. The interconnected boreholes should be backfilled with a pervious drainage material and sealed at the top with an impermeable soil cover to prevent infiltration of surface water. Gravity drainage is provided through connecting horizontal drains. A completed system of vertical and horizontal drains should be checked by monitoring embankment deformations, water discharge rate, and changes in groundwater levels and pore-water pressures. Remedial treatment methods in addition to drainage should be implemented if the embankment does not stabilize or a higher factor of safety is required.

257. Vertical drain backfill should consist of clean, coarse aggregate with filter protection provided as necessary (paragraphs 242 to 243).

Filter protection generally will not be needed for portions of drains located in competent sedimentary rock foundations. In some sidehill fill applications, sections of vertical drain may pass through the shale fill or underlying residual soils. These sections of drain should be provided with filter protection to prevent infiltration of adjacent materials. Filter protection will be necessary in soil foundations. A circumferential filter layer can be installed by placing a hollow pipe inside the vertical borehole and backfilling between the pipe and borehole wall with filter aggregate. The space inside the pipe should be filled with coarse drainage aggregate as the pipe is removed from the borehole.

258. Vertical drains should be deep enough to allow interception by horizontal drains initiated near the toe of the fill (see Figure 50). Horizontal drain holes should be drilled toward the belled-out zones. Intercepting the belled-out zones with horizontal drains is difficult because of borehole deviation during drilling; however, because the vertical drains are interconnected, only 2 to 4 horizontal drains per 10 vertical drains are normally required. When vertical drains are not interconnected, intercepting individual vertical drains with horizontal drains may be difficult at distances much greater than 100 ft. Where vertical drains cannot be intercepted by horizontal drains, gravity drainage may be possible if the vertical drains can be drilled to an underlying, free-draining (or highly permeable), foundation layer below the base of the embankment.

259. Vertical drains have also been used to drain water accumulated (through surface infiltration) within porous zones (e.g., limestone or sandstone layers) formed during initial embankment construction. However, their application is limited to the situation shown in Figure 51 where (a) the vertical drains will pass through the lowest portion of the porous embankment layer and (b) gravity drainage can be provided by intercepting an existing culvert at the base of the fill as described by Clark (1976).* Because of these limitations, horizontal drains will be more convenient in most situations for draining porous layers contained within shale fills. If vertical drains are used within shale fills, filter protection should be provided to prevent clogging with fine-grained soils that are normally contained within the fill or which could be generated during long-term erosion of the shale materials.

260. Pumped vertical wells. Rapid lowering of the groundwater level can be achieved by using pumped vertical wells. Because of maintenance and energy requirements, pumped vertical wells should be applied

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* Clark, P. C. et al., "Vertical Drains for Highway Embankments in Kansas," Soil Mechanics: Rutting in Asphalt Pavement, Embankments on Varved Clays, and Foundations, Transportation Research Record 616, Transportation Research Board, Washington, D. C., 1976.

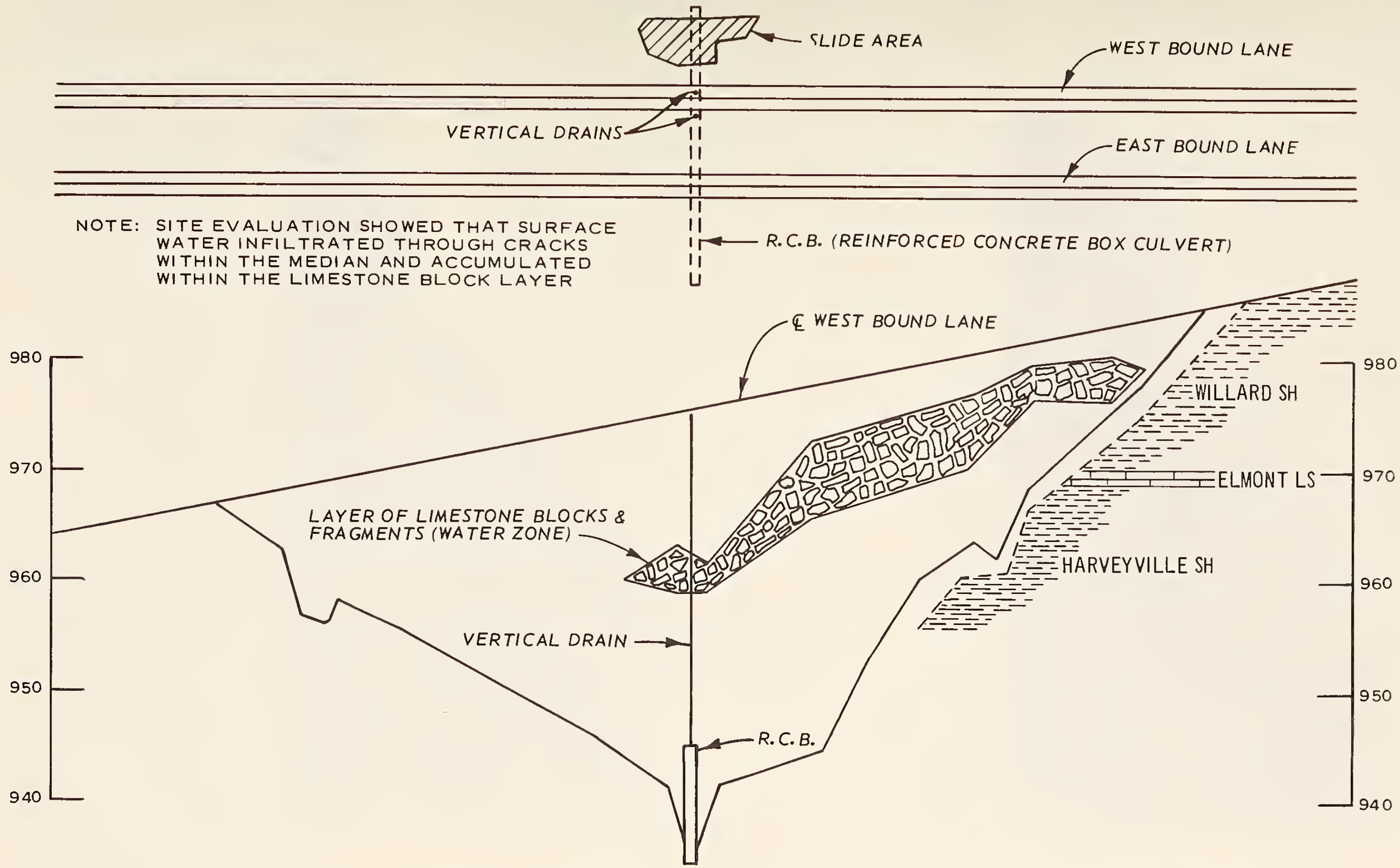
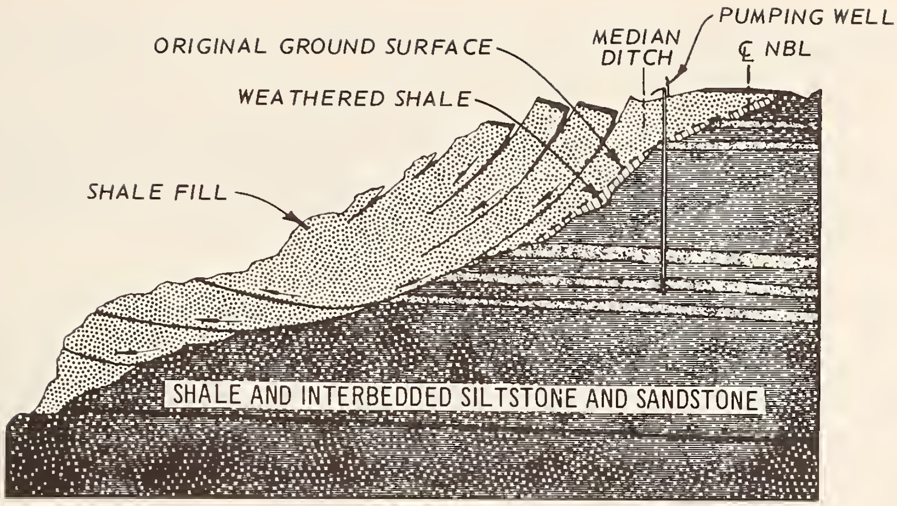


Figure 51. Vertical drains used to drain water accumulated in limestone layer (Clark et al., 1976)

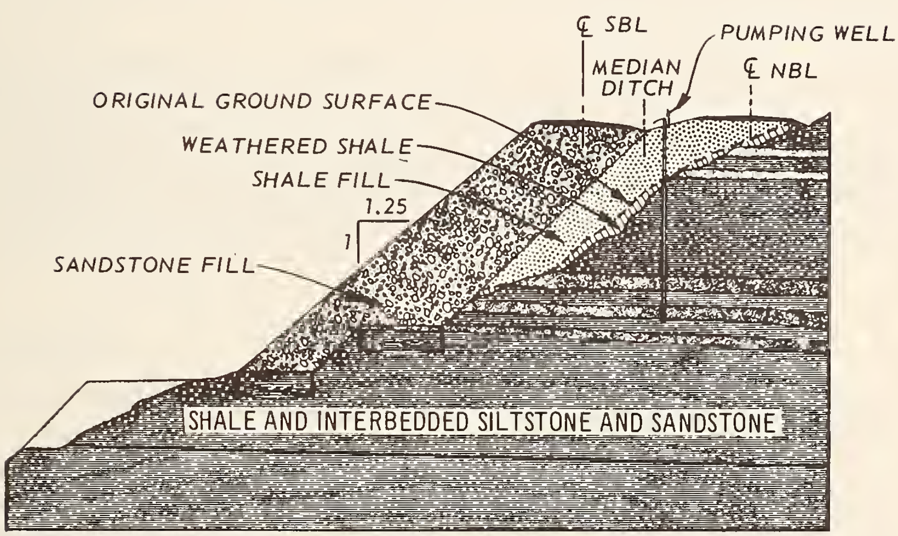
only as a temporary remedial measure. Pumped wells should be considered primarily for maintaining stability of excavations required in removal and replacement of failed embankment slopes, construction of shear trenches or rock buttresses, or construction of retaining walls. Pumped wells should also be considered for maintaining embankment stability during rainy seasons until permanent remedial treatment can later be applied. Pumped vertical wells were used to maintain stability during reconstruction of a failed slope on a sidehill shale embankment on I-75 in Tennessee. Twelve 100-ft-deep vertical wells were drilled on 25-ft centers through the median above the slide, as shown in Figure 52. Each well consisted of a 6-in.-diameter slotted casing, a 1/2-hp submersible pump, 1/2-hp pump with automatic operation controls, and connecting discharge pipe.

261. In treatment of sidehill fills, it is recommended that pumped vertical wells be placed behind the fill to collect subsurface seepage prior to entrance to the shale embankment. Wells having 6- to 12-in. diameters and spaced on 20- to 30-ft centers should be satisfactory in most sidehill fill applications. Discharge from systems of pumped vertical wells should be directed away from the embankment slope. Portions of wells within shale fill or soil foundations will require filter protection to prevent infiltration of adjacent materials. Filter protection can be provided by surrounding the casing with fabric filter and/or filter aggregate (paragraphs 242 to 243). Specialized, commercially available well screens can also be used. In competent sedimentary rock strata, slotted or perforated casing without added filter protection will normally be satisfactory. Embankment deformation, well discharge rate, and changes in ground water levels and pore-water pressures should be monitored to check the effectiveness of the well systems. Operation of wells should be checked periodically and repairs made as necessary. Guidance for planning, design, construction, and operation of pumped vertical wells is contained in Groundwater and Wells (Edward E. Johnson, Inc., 1972) and the Technical Manual "Dewatering and Groundwater Control for Deep Excavations," TM 5-818-5 (Departments of the Army, Navy, and Air Force, 1971).

262. Interceptor trench drains. A continuous trench installation designed and oriented to collect subsurface seepage is termed an interceptor trench drain. In past applications, interceptor trench drains have been installed in the natural slope behind sidehill fills to collect subsurface seepage entering the embankment and residual soil foundation. These drains are generally ineffective probably because of their shallow depth (10 ft maximum). Deeper trenches would be more effective; however, excavation costs are generally prohibitive except when constructing shear trenches. Shear trenches have an important function as interceptor trench drains, but are located within the embankment and/or foundation to counteract known or potential shear failure (paragraphs 271 to 273). A more effective method of intercepting subsurface seepage toward shale fills is a row of closely spaced, interconnected, vertical drains (paragraphs 255 to 259).



a. EMBANKMENT FAILURE



b. EMBANKMENT REPAIR

Figure 52. Failure and repair of shale embankment at sta 840+00, I-75, Tennessee (Royster, 1973)

263. Interceptor trench drains are most effective when placed as embankment toe drains, as shown in Figure 53 to improve and/or maintain toe stability. Stability of the embankment toe is critical, since a decrease in toe support can lead to progressive failure of the embankment slope. In Utah, toe drains have been used adjacent to embankments constructed of the Mancos Shale. Utah State Highway Department soils engineers consider the toe drains to be a key factor in maintaining embankment stability.

264. It is recommended that interceptor trench drains be limited to depths of 5 to 10 ft and used primarily as embankment toe drains; however, whether used behind shale fills or as toe drains, additional drainage measures (such as horizontal drains) and/or other remedial treatment methods are recommended. Interceptor trench drains should contain a perforated collector pipe embedded in clean, coarse drainage aggregate and leading to an outlet at the end of the trench or one or more lateral outlet drains. The coarse backfill should partially fill the trench with a minimum requirement of blanketing the trench bottom and backslope as shown in Figure 53. Filter aggregate or filter fabric should be placed along the trench bottom and backslope to prevent infiltration of soil fines from the adjacent embankment and/or foundation (paragraphs 242 to 243). The trench should be topped with an impervious soil cover to prevent infiltration of surface water. If deep toe drainage is critical, shear trenches should be constructed adjacent to the embankment toe (see paragraphs 271 to 273).

265. Trench wall stability must be maintained during trench excavation and backfilling. Toe excavation is particularly hazardous since a decrease in toe support can cause instability of the embankment slope. Excavation stability during construction of trenches in the 5- to 10-ft maximum depth range is not likely to cause significant problems; however, measures such as those recommended in shear trench construction should

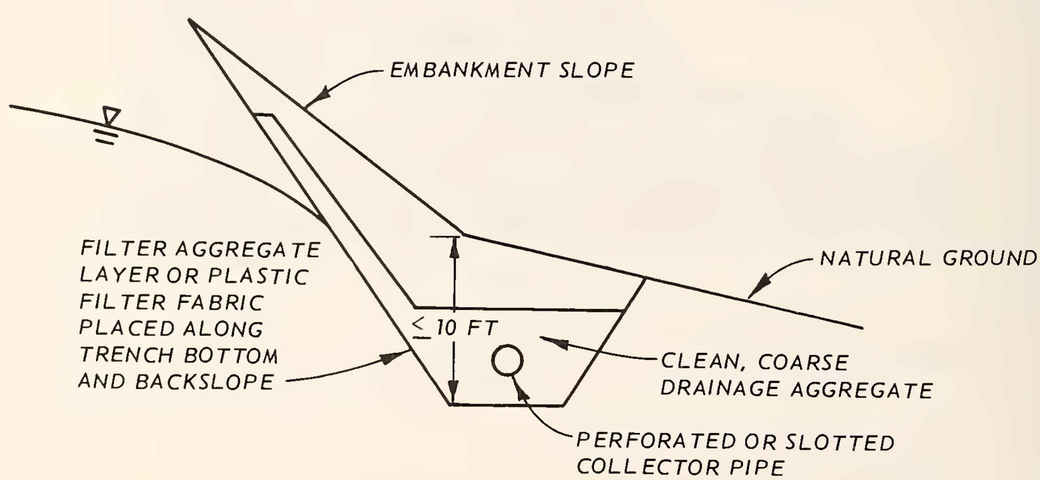


Figure 53. Interceptor trench-embankment toe drain

be implemented, as necessary, to maintain stability (see discussion, paragraph 274).

266. Drainage blankets. A drainage blanket consists of a layer of pervious drainage materials designed for collection and control of subsurface seepage. In treatment of shale embankments subject to underground seepage, drainage blankets should be placed beneath:

a. Fill added to flatten slopes and/or construct berms (paragraph 270).

b. Backfill placed in shear trenches (paragraph 273).

c. Backfill placed behind retaining walls (paragraph 280).

d. Replacement fill for embankment reconstruction (paragraph 287).

Applications and recommended features of drainage blankets are summarized in Figure 54. Drainage blankets should be constructed of clean, coarse aggregate. The coarse aggregate should not be placed directly adjacent to erodible foundation material or shale fill. A layer of filter aggregate or plastic filter fabric should be used to prevent infiltration of fines and subsequent clogging of the coarse aggregate layer (paragraphs 242 to 243). When fill material overlaying the drainage blanket consists of compacted soils or shale material, a minimum 3-ft-thick blanket with filter protection should be used. When fill material consists of clean sand and/or gravel mixtures or nondegradable rockfill, the drainage blanket thickness can be reduced (or the drainage blanket can possibly be eliminated). However, a thin layer of filter aggregate or plastic filter fabric should always be included. Collected water should be removed through a perforated or slotted collector pipe (or pipes) embedded within the drainage blanket.

Slope-flattening, berms, and shear trenches

267. Slope-flattening, berms, and shear trenches have been common and reliable methods for treating slope instability problems in shale embankments. These techniques should be given primary consideration when treatment in addition to (or other than) drainage methods is required to improve stability. The economy and feasibility of constructing flatter slopes, berms, and shear trenches depends primarily on site topography, adequate right-of-way, and availability of suitable borrow materials. Additional and more important factors in shear trench feasibility are trench excavation depth and stability requirements.

268. Flatter slopes and/or berms are conveniently constructed by placing fill on the distressed slope, as shown in Figure 55, after regrading and removal or drying of saturated materials. This placement

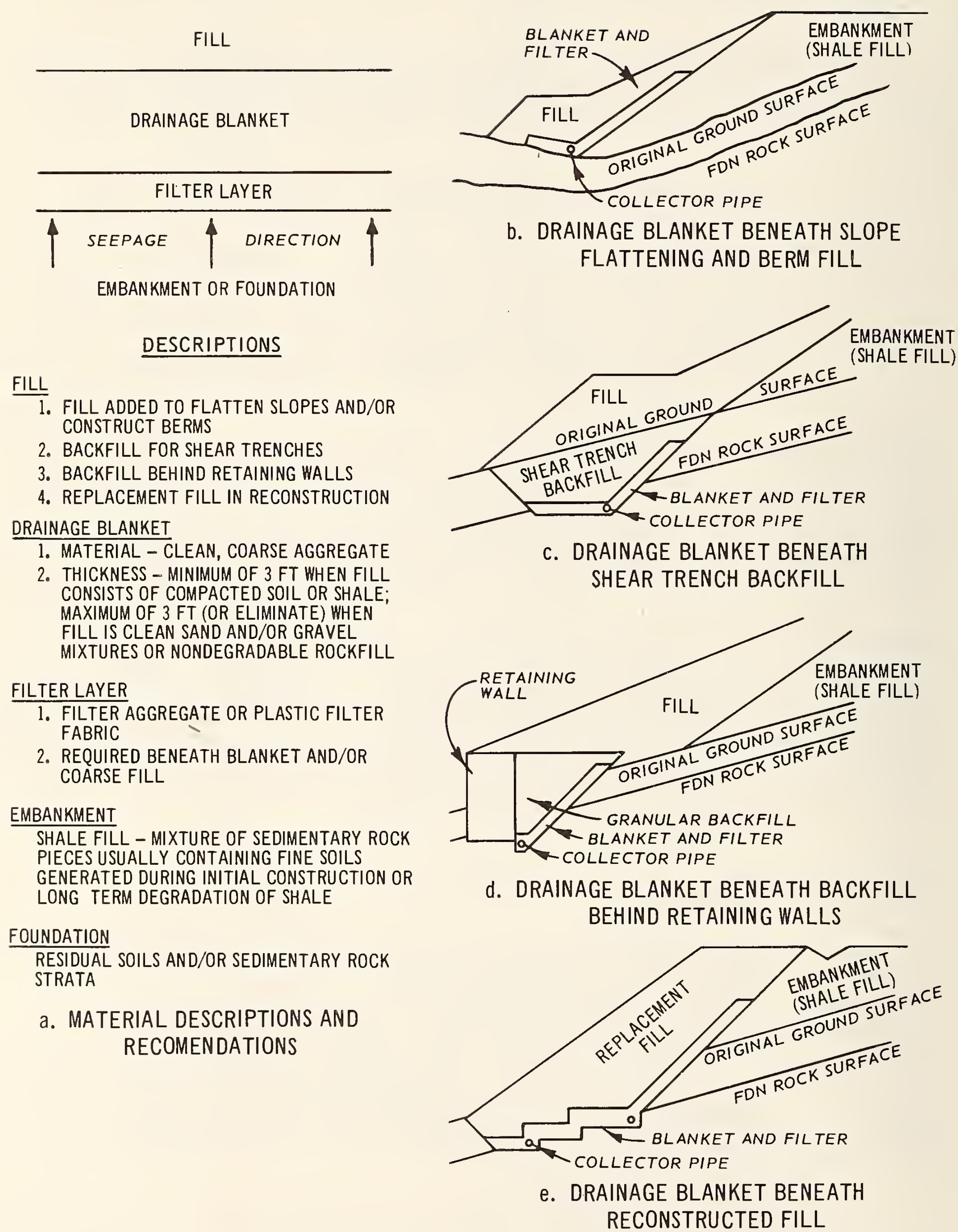


Figure 54. Applications and recommended features of drainage blankets

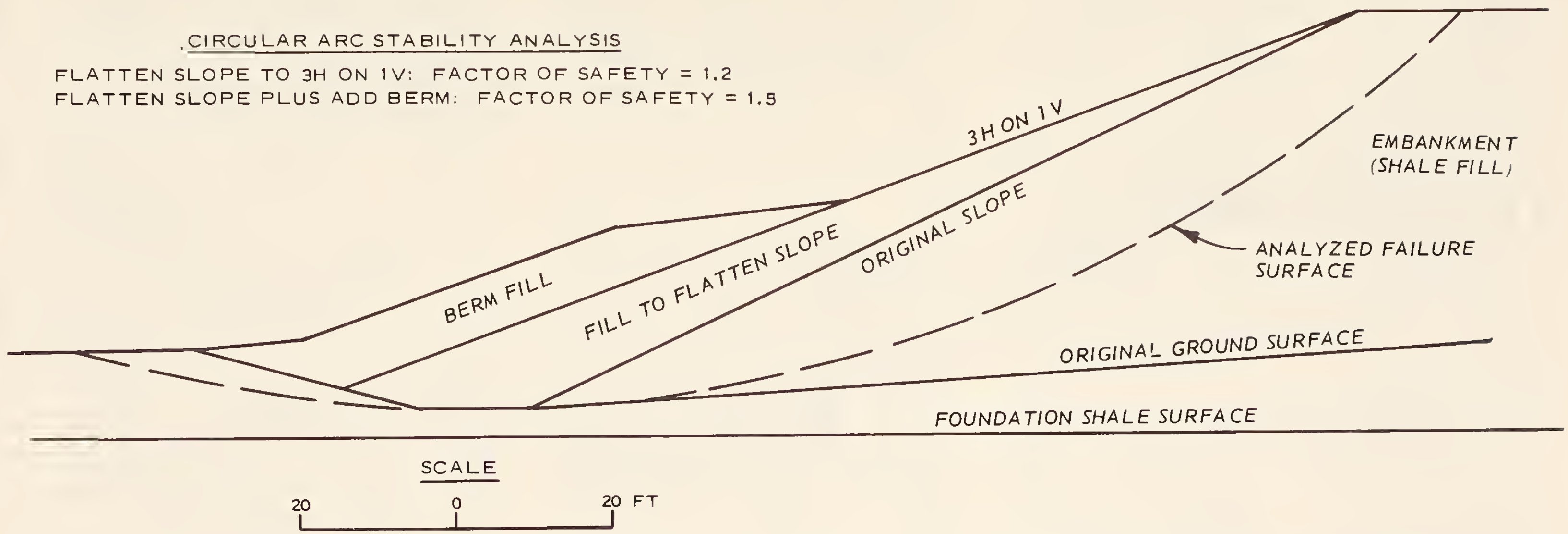


Figure 55. Comparison of theoretical factors of safety for slope-flattening with and without an added berm for remedial treatment of a shale fill, milepost 152.7, sta 3481+00, I-64, Kentucky (courtesy of the Kentucky Department of Transportation)

procedure causes minimal disturbance to the existing embankment and highway. Flatter slopes and berms can also be incorporated into reconstructed fills. Combined slope-flattening and berm fills (i.e., stabilization fills) are generally significantly more effective and practicable than slope-flattening alone. For example, theoretical stability analysis of the flattened slope shown in Figure 55 resulted in a factor of safety of 1.2. The addition of the berm fill shown in Figure 55 resulted in a factor of safety of 1.5.

269. Soil or rock material can be used to flatten slopes and construct berms. The type of fill selected should be determined primarily by acquisition and placement costs. It is recommended that coarse, nondegradable rockfill (compacted during placement) be used when feasible; however, limited availability of suitable rockfill in many areas will likely confine its use to primarily low volume rock berms (or buttresses). Soils and shale materials will be most common in slope-flattening and berm construction. These materials may require compaction during placement to attain adequate density and shear strength.

270. The construction of flatter slopes and/or berms should be supplemented with subsurface drainage measures when treating shale embankments subject to underground seepage. These embankments will include primarily sidehill and transitional fills. The minimum requirement is placement of a drainage blanket beneath the added fill (see Figure 54 and discussion, paragraph 266). Stabilization fills and drainage blankets used in remedial treatment of two embankment slides located on I-74, Indiana, are shown in Figures 49 and 56. Although not shown in Figure 49, horizontal drains were also used to supplement the berm and drainage blank (see discussion, paragraph 251). The berms were constructed of shale materials broken down and compacted in thin lifts. A series of 12-ft-deep, 6-ft-wide benches (not shown in Figures 49 and 56) were excavated into the shale embankments to facilitate construction of the drainage blankets and berms.

271. Slope instability in shale embankments often involves shear failure along the fill-foundation contact or through weak residual soil or weathered shale foundation layers. Slope-flattening and berms are often supplemented with shear trenches (or shear keys) to counteract known or potential foundation shear failures. Generally, shear trenches are located adjacent to the embankment toe as shown in Figure 57. This location is advantageous in that shear resistance is added to the critical embankment toe area and shear trench construction does not require excavation of the existing embankment. A large portion of the berm fill should be placed directly over the shear trench to develop significant normal stress and frictional shear resistance within the trench backfill. Excavation costs and difficulties generally prohibit placement of shear trenches directly beneath and/or within the embankment slopes. However, these locations should be considered when large-scale embankment reconstruction is to be conducted.

272. Shear trenches should extend the entire embankment length,

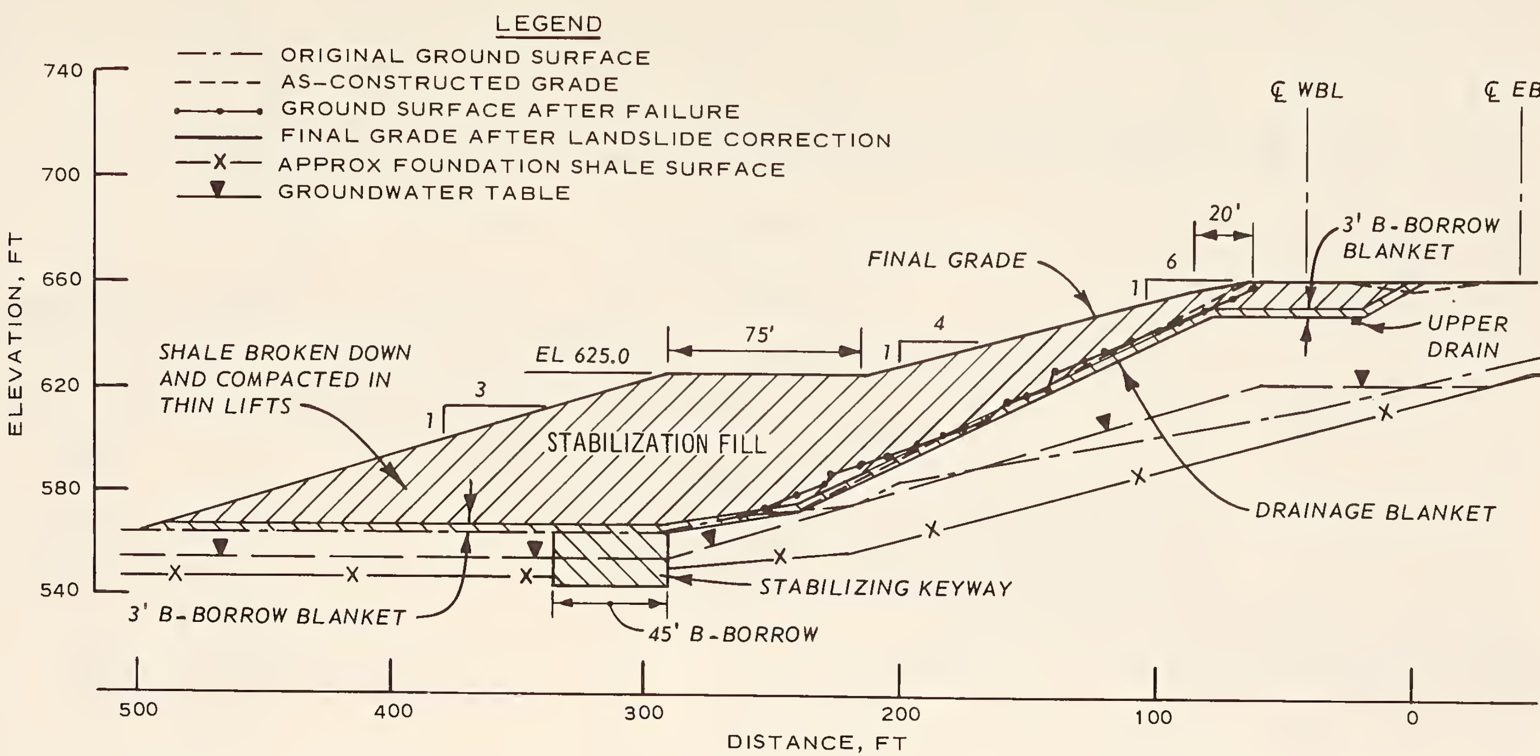


Figure 56. Remedial treatment of a shale embankment slide along WBL, I-74, Indiana (courtesy of the Indiana State Highway Commission)

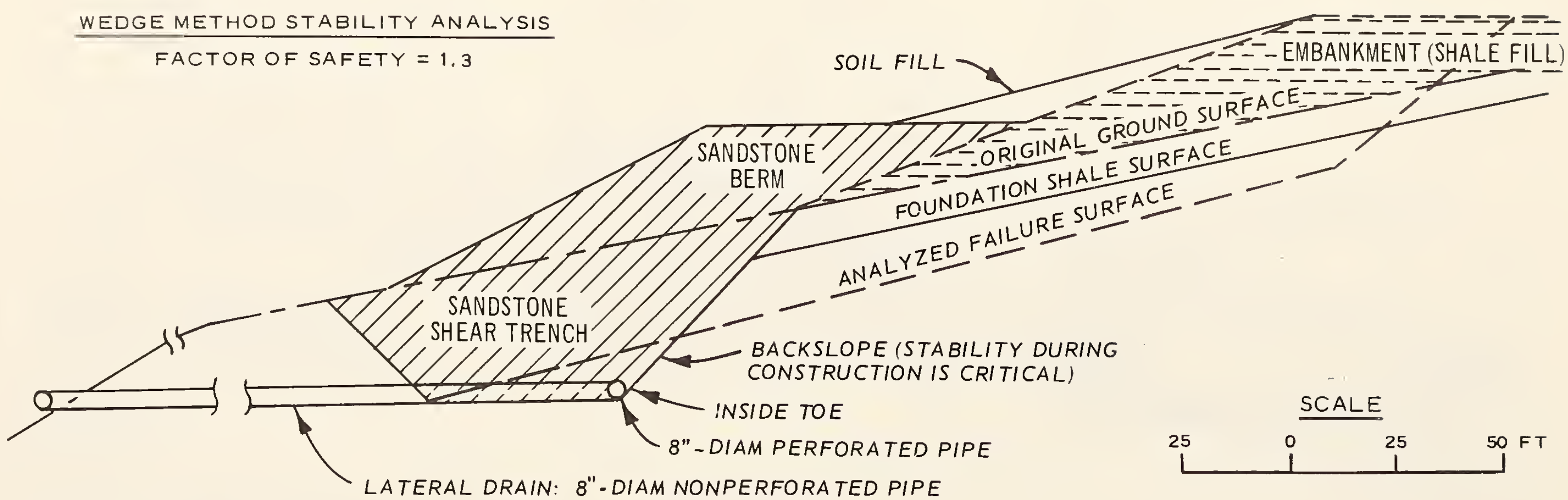


Figure 57. Remedial treatment of shale fill at milepost 92, sta 6192+50, Western Kentucky Parkway, Kentucky (courtesy of the Kentucky Department of Transportation)

be keyed into firm foundation rock, and be backfilled with coarse, non-degradable rock compacted in place. For example, the shear trench shown in Figure 57 was 1200 ft long, keyed into shale, and backfilled with sandstone placed in 1-ft lifts and compacted with a vibratory compactor. If necessary, soil or shale fill can be used in shear trenches, provided that significant shear strength can be achieved through proper compaction during fill placement. Benching, moisture control, thin lifts, and special procedures to break down shale materials should be used to obtain adequate compaction (see discussion of compaction, Part VI).

273. In addition to providing increased shear resistance, shear trenches have an equally important function as subsurface drains. Recommended design of shear trenches used for collecting groundwater seepage is given in Figure 58. The trench bottom should slope from outside toe to inside toe at approximately a 3- to 5-percent grade (except at locations of lateral outlet drains). A drainage blanket should be placed along the trench bottom and backslope (see Figure 54 and discussion, paragraph 266). A perforated collector pipe should be placed along the inside toe of the trench to collect seepage water and carry it to an outlet at the end of the trench or one or more lateral outlet drains. Lateral outlet drains consist of buried nonperforated pipes. For example, the shear trench cross section shown in Figure 57 corresponds to a lateral drain location. Lateral drains were placed at each end and one in the central portion of the 1200-ft-long shear trench. In certain cases, estimated seepage rates will require that perforated collector pipes also be placed laterally on the trench backslope at intermittent locations along the trench.

274. Shear trenches can normally be constructed with trench walls

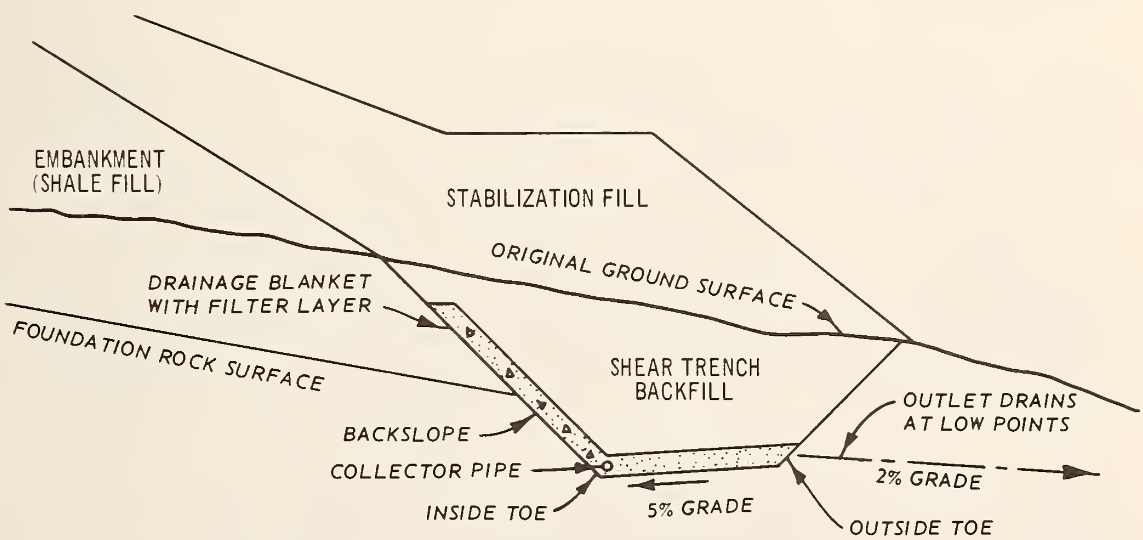


Figure 58. Recommended design of shear trenches for collecting groundwater seepage

ranging in steepness from 1V on 1H to 1V on 1.5H. However, flatter slopes should be used, when necessary, to maintain trench wall stability during excavation and backfilling. Stability of the trench backslope (see Figure 57) is most critical, since backslope failure could cause further instability of the adjacent shale embankment. If necessary, pumped vertical wells should be installed behind the proposed trench backslope to lower the groundwater table during trench excavation and backfilling. It is recommended that, when feasible, shear trenches be constructed during a dry season of the year and long trenches be completed in 50- to 100-ft alternate segments, depending on existing backslope stability. Construction of each trench segment should proceed in a continuous 24 hour/day operation. Shear trench excavation has generally been economically feasible when maximum depth (i.e., depth at inside toe, Figure 57) was less than 30 to 40 ft. The shear trench shown in Figure 57 was 1200 ft long, 10 to 30 ft deep, and constructed during a dry season by completing 100-ft-long segments. Construction of each 100-ft segment proceeded in a continuous 24 hour/day operation.

275. Theoretical stability analyses should be included in the design of all remedial treatment slopes, berms, and shear trenches. Stability analyses should consider existing and potential shear failure surfaces through the existing shale embankment and foundation, slope and berm fill, and shear trench. Stability of berm slopes should also be analyzed separately and stability analyses should be used to determine safe slope angles for temporary shear trench excavations. Groundwater conditions should be considered in stability analyses to determine the extent of subsurface drainage required to provide permanent (or temporary) stability (see discussion of stability analyses, paragraphs 130 to 134 and Vol. 2).

Retaining walls

276. Retaining walls should be considered primarily for support of slope-flattening and berm fills or incorporation into shale embankment reconstruction at locations where right-of-way and/or borrow material is limited. Retaining walls considered most appropriate are reinforced earth, crib, and gabion retaining walls. Concrete retaining walls should also be considered; however, applications will be limited because of foundation requirements and construction costs. Types, design, and construction of concrete retaining walls are discussed in the textbook Foundation Analysis and Design and Analytical and Computer Methods in Foundation Engineering (Bowles, 1968 and 1974) and the design manual "Soil Mechanics, Foundations, and Earth Structures," NAVFAC DM-7 (Department of the Navy, 1971). Reinforced earth, crib, and gabion retaining walls are generally less expensive than concrete retaining walls and can withstand significant total (or nonuniform) vertical and lateral movement without loss in stability. Another type of slope retaining method applicable under certain conditions is closely spaced piles installed in one or more rows across the embankment slope. Features and recommendations concerning reinforced earth, crib, gabion, and pile retaining methods are presented in subsequent paragraphs.

277. Reinforced earth, crib, and gabion walls. The basic components and recommended features of reinforced earth, crib, and gabion walls are shown in Figures 59, 60, and 61, respectively. The reinforced earth* retaining wall (Figure 59) consists of three major components: the backfill material, the reinforcement strips, and the skin (or facing) units. The backfill material should consist of granular material, such as sand and gravel with less than 15 percent (by weight) passing the No. 200 sieve. The maximum particle size of the backfill should not exceed 10 in. The backfill should be placed and compacted in thin horizontal layers. The reinforcement strips normally consist of thin, wide, long strips that have a rough surface and high tensile strength (e.g., metal, fiberglass, etc.). Reinforcement strips of galvanized steel or special materials are commonly used to prevent corrosion. The reinforcement strips are regularly spaced in both the horizontal and vertical direction. The skin or facing units are used to protect the surface of the wall and to prevent raveling of the backfill material. The skin or facing units can be constructed of metal plates, curved metal sheets, or reinforced concrete panels. Reinforced earth retaining walls are normally constructed at height-to-width ratios of 0.8 to 1.0. In highway construction or remedial treatment, reinforced earth walls have generally been limited to heights of 20 to 50 ft; however, reinforced earth walls have been constructed to a maximum height of 85 ft.

278. A crib wall consists of a series of interconnected cells (commonly 8 to 12 ft square) constructed of wood, metal, or precase reinforced concrete struts, as shown in Figure 60. Crib cells should be filled with granular material, such as mixtures of crushed rock, sand, and gravel. Cell fill should be placed in thin lifts and compacted as the wall is constructed. Crib walls greater than 12 ft in height should be battered toward the supported fill at a slope of 6V on 1H (i.e., 2 in. per ft of height) and supplemented with lower cribs placed behind the main wall. Loose soil should not be allowed to spill onto the top of the granular cell fill because during rains the loose soil could wash down into the granular fill and cause clogging.

279. A gabion wall is constructed of interconnected rectangular wire-mesh containers filled with nondegradable rock, as shown in Figure 61. The wire mesh consists of 11-gauge galvanized steel wire woven in a hexagonal pattern with openings of about 3 to 4 in. The gabion containers are nominally 1, 1.5, or 3 ft square on the ends and are available in 6-, 9-, and 12-ft lengths. Fill material should consist of hard, durable stones placed and compacted in 1-ft lifts. The gabion containers are constructed side-by-side and atop one another to form a retaining wall. The gabion containers are laced together along the

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* The reinforced earth concept and methodology described in this manual was patented by Henri Vidal (U. S. Patent No. 3,421,326, Jan 14, 1969) and the Reinforced Earth Co., Washington, D. C., is the U. S. licensee.

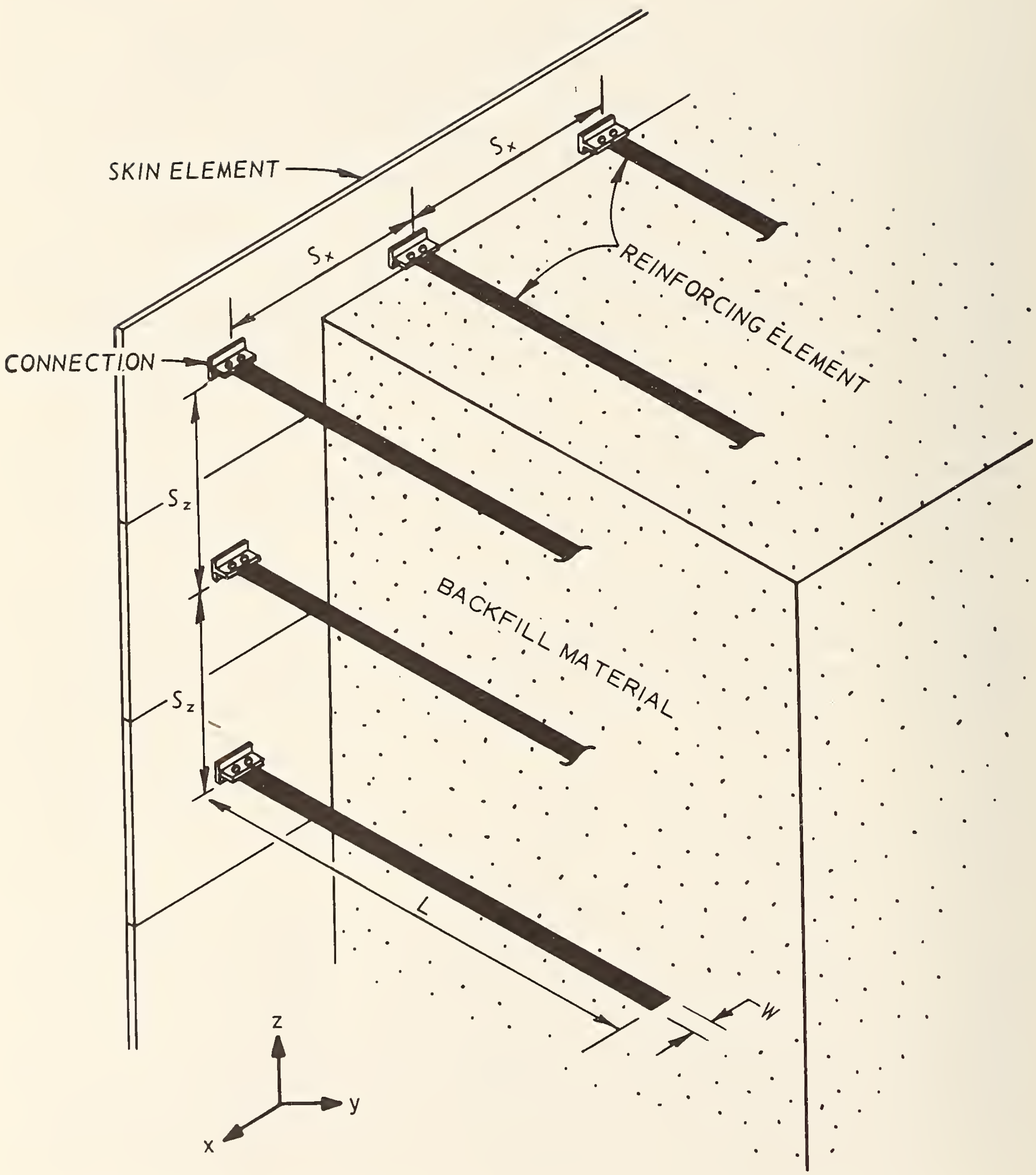

Figure 59. Schematic of major elements of reinforced earth wall (Al-Hussaini and Perry, 1976)

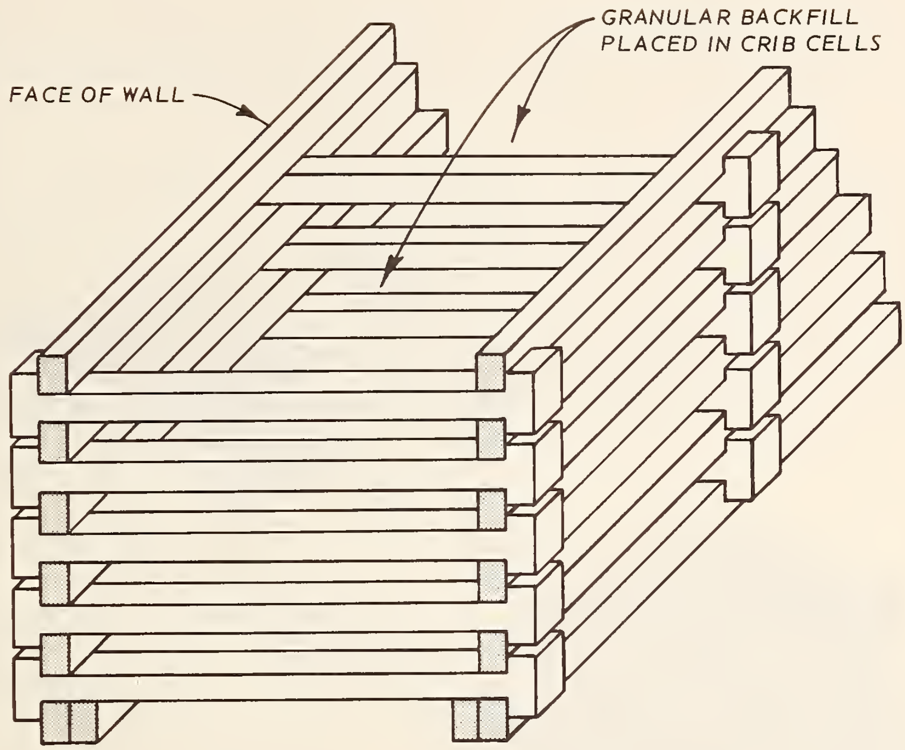

Figure 60. Components of a crib retaining wall

perimeter of all contact surfaces using 13-gauge steel wire. Gabion walls should be battered toward the supported fill at a slope of 6V on 1H (i.e., 2 in. per ft of height). Experience and problems with gabion walls are described by Larsen (1975)* and Blackburn (1973).**

280. Reinforced earth, crib, and gabion walls should be embedded a minimum depth of 10 to 20 percent of the wall height and founded on firm rock, in situ soil, or compacted embankment fill. It is recommended that, when feasible, residual soils be excavated to provide a firm unweathered rock foundation when retaining walls are constructed adjacent to shale embankments. Measures such as those recommended in shear trench construction should be implemented as necessary to

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* Larsen, R. E., "Seismic Designed Backslopes and Evaluation in a Structurally Disturbed Basalt Section," 26th Annual Highway Geology Symposium Proceedings, Idaho Department of Transportation, Division of Highways, Boise, ID, Aug 1975.

** Blackburn, J., "Gabion Construction on Slides at Interstate 40 Near Rockwood, Tennessee," Proceedings of the 54th Annual Tennessee Highway Conference, Bulletin No. 39, Engineering Experiment Station, The University of Tennessee, Knoxville, TN, Jan 1973.

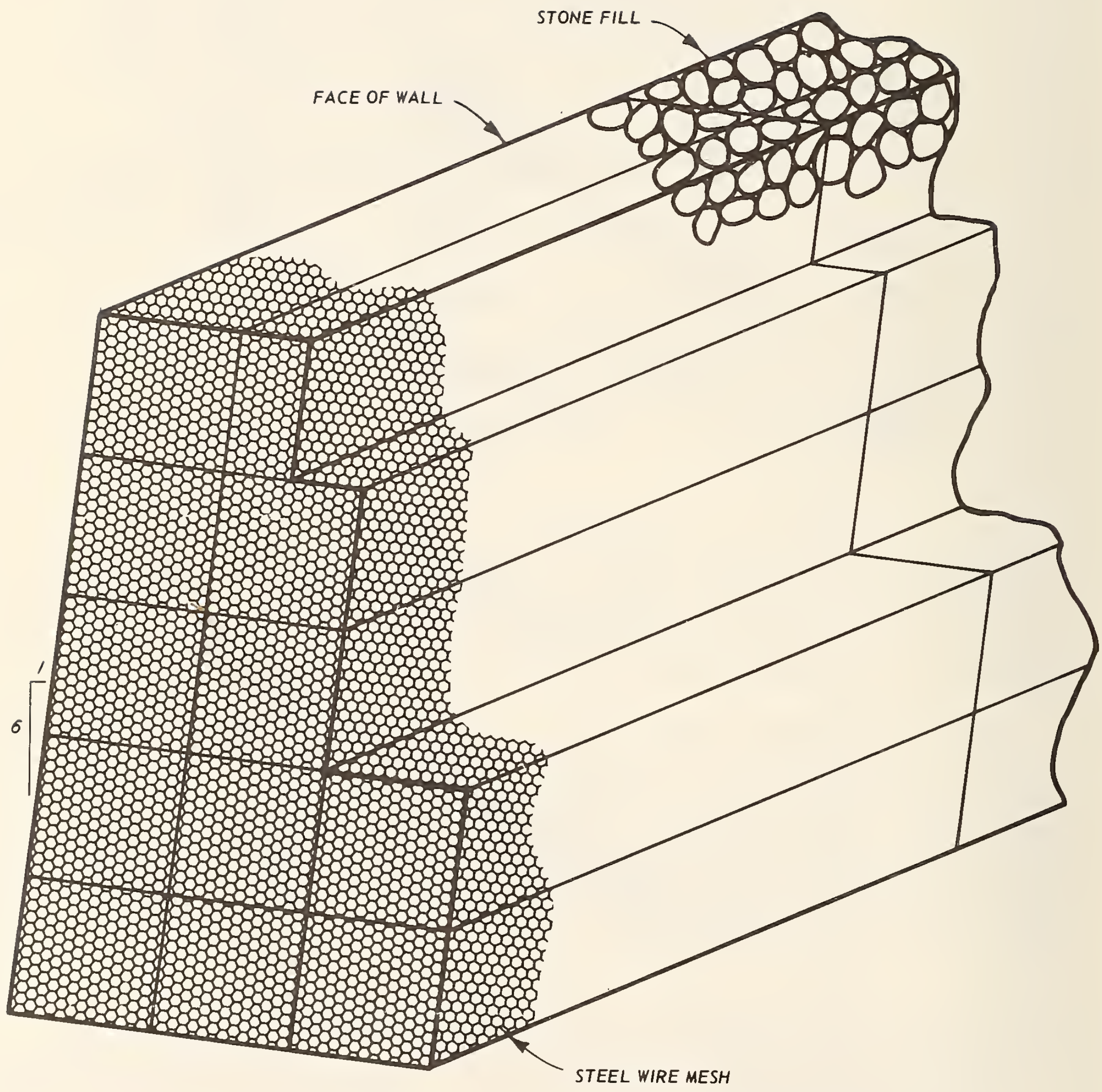

Figure 61. Components of a gabion retaining wall

maintain stability of excavations required during wall construction (see discussion, paragraph 274). A limited zone of granular backfill underlaid with a drainage blanket should be placed behind retaining walls to prevent excessive buildup of pore-water pressure and ponding of water along the wall foundation (see Figure 54 and discussion, paragraph 266).

281. Selection of reinforced earth, crib, and gabion retaining walls should be based on primarily local construction experience, availability of construction materials, construction time frame, and overall costs. There are, however, certain situations where a specific type of retaining wall may prove advantageous. Gabion walls provide effective erosion control and should be given greater consideration for support of remedial slopes and/or berms adjacent to or penetrating rivers and streams. When wall heights greater than 20 ft are required, reinforced earth walls are likely to be the most effective and economical alternative.

282. Reinforced earth, crib, and gabion walls should be designed and analyzed as gravity retaining structures. Design analyses should consider essentially (a) internal failure, (b) overturning failure, (c) bearing capacity failure, and (d) slope shear failure. Slope stability analyses should consider groundwater conditions and potential shear failure surfaces through the supported fill, foundation, and the retaining wall itself (see discussion of stability analyses, paragraphs 130 to 134). Analytical concepts and design of reinforced earth retaining walls are presented in the WES report "Effect of Horizontal Reinforcement on Stability of Earth Masses," TR S-76-11 (Al-Hussaini and Perry, 1976) and by Schroeder et al. (1976).* The effect of corrosion is reviewed by King (1977).** Design guidance for reinforced earth is also available from the Reinforced Earth Co., Washington, D. C. Design and analysis of crib walls is presented in the design manual "Soil Mechanics, Foundations, and Earth Structures," NAVAFAC DM-7 (Department of the Navy, 1971). Further discussion of crib wall design and behavior is contained in the textbooks Foundations, Retaining and Earth Structures (Tschebotarioff, 1973) and Foundations Design and Practice (Seelye, 1956). The basic design concepts presented for crib walls in the aforementioned references are applicable to gabion walls. Design guidance for gabion walls is also obtainable from companies manufacturing and selling the gabion wire containers.

283. Pile rows. A rapidly applied method of adding shear support

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* Schroeder, W. L. et al., "Performance of a Thin Metal Retaining Wall with Multiple Anchorage," Transportation Research Record 616, Transportation Research Board, Washington, D. C., 1976.

** King, R. A., "A Review of Corrosiveness with Particular Reference to Reinforced Earth," Transportation and Road Research Laboratory, Supplementary Report 316, Berkshire, England, 1977.

to an unstable embankment slope is to construct a row (or rows) of closely spaced vertical piles across the embankment slope. Piles are most conveniently installed along the highway shoulder; however, additional piles placed, for example, along the embankment toe may be necessary for permanent stability. Critical factors in pile support capability are pile bending strength, spacing, depth, and anchorage. To be effective, vertical piles must be closely spaced (normally less than 3 ft), cross the existing or potential shear zone, and be anchored in competent rock as shown in Figure 62. In remedial treatment applications, it is recommended that primary consideration be given to using steel piles placed in predrilled boreholes backfilled with concrete. Less costly reinforcement and backfill can be considered for temporary (maintenance type) support or treatment of shallow slides. For example, the Kentucky Department of Transportation has successfully used railroad rails placed in predrilled boreholes (spacing less than 3 ft) and backfilled with concrete, sand, or pea gravel in treating shallow landslides (depth to competent rock less than 20 ft).

284. Retaining walls can be constructed by placing wood planks, wood piles, steel guardrails, or precast concrete panels against exposed pile sections. A common maintenance practice has been to install piles adjacent to distressed shoulder areas, construct a retaining wall using old guardrail or wood planks, and place backfill to repair or extend the shoulder. Although this practice may be necessary to maintain adequate shoulder width, it should be realized that the backfill in the shoulder area adds a driving shear force that can offset the beneficial effects of the piles. Pile retaining walls constructed at or near the embankment toe provide support for slope-flattening or berm fills, as shown in Figure 62. The disadvantage is that periodic dumping of material to maintain the shoulder area can eventually increase the driving force above the design value for pile walls near the toe. In treatment of sidehill fills it is recommended that, when feasible, the retaining wall be anchored to competent rock as shown in Figure 63. Anchorage is provided by posttensioning steel cables (or rods) placed in boreholes and grouted into the rock.

285. Whether used for temporary or permanent support, pile systems should be designed and analyzed to determine their support capabilities. In past practice, highway maintenance crews have often installed piles without prior site evaluation or stability analyses. The effectiveness of these pile installations has been marginal. When feasible, maintenance personnel should obtain recommendations from the geotechnical engineering staff before installing piles. For example, when treating sidehill fills, subsurface drainage (using, for example, horizontal drains) is likely to be a necessary supplement or alternative to pile installations. Where maintenance crews have installed piles without prior analysis, a subsequent evaluation should be conducted by geotechnical engineers to determine the capabilities of the pile system and need for additional treatment. A method of analyzing the stabilizing effect of pile rows is presented in Transportation Research Board Special Report

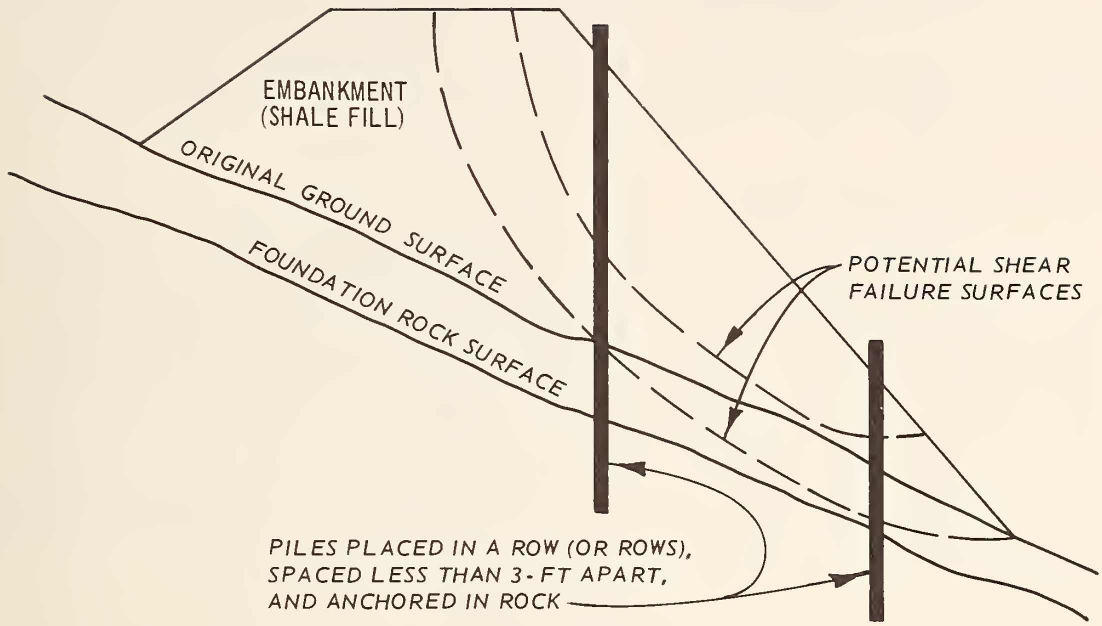

Figure 62. Piles located to provide lateral restraint against shear failure

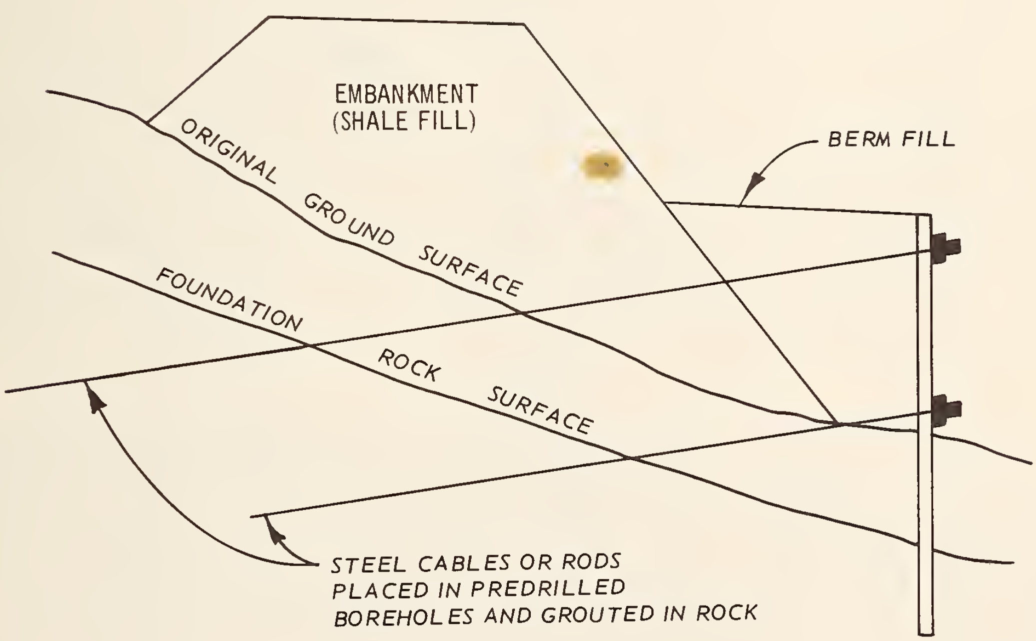

Figure 63. Anchored pile retaining wall to support berm fill

No. 176, Landslides; Analysis and Control, edited by Robert L. Schuster and Raymond J. Krizek, 1979. Modes of failure considered in the analysis are (a) overturning, bending, or shear through the piles; (b) shear failure beneath the piles; and (c) shear between the piles. Stability analyses are used to compute lateral earth forces on the piles for an assumed shear surface.

Embankment reconstruction

286. Embankment reconstruction involving the removal and replacement of shale fill (and foundation) materials has been an effective, but costly, remedial treatment method. Reconstruction should be considered primarily when large settlements, shear displacements, and/or shale degradation have significantly weakened the fill or foundation materials. Remedial treatment methods previously discussed (i.e., surface and subsurface drainage, slope-flattening and berm fills, shear trenches, and retaining walls) should be incorporated into reconstruction plans, as necessary, for effective treatment. Thorough site evaluation (Part III), including stability analyses, are necessary to determine the extent and feasibility of reconstruction plans.

287. In embankment reconstruction, materials should be removed to a depth well below the known (or potential) shear zone or failure surface. In treatment of sidehill fills, it is recommended that residual foundation soils be removed, benches cut into the foundation rock, and a drainage blanket placed before reconstructing the fill (paragraph 266). In repair of a slide along I-74, Indiana, unstable shale fill and residual foundation soils were removed and the embankment reconstructed. Drainage blankets and collector pipes were placed on the foundation bench slopes, as shown in Figure 64. It is recommended that in most applications drainage blankets extend across the rock benches. Removal of materials may be confined to the shale fill when either (a) shear failure and shale degradation are isolated near the slope surface, (b) foundation shear strength is substantial, or (c) the foundation residual soil layer is too extensive for economical excavation. In these cases, the drainage blanket should be placed directly on the excavated shale slope, and supplementary drainage methods (for example, horizontal drains) should be applied.

288. Replacement fill in embankment reconstruction can consist of soil or rock material. The type of fill selected should be based primarily on shear strength requirements and availability. It is recommended that coarse, nondegradable rockfill (compacted during placement) be used when feasible; however, limited availability of suitable rockfill in many areas will likely limit its use to primarily low volume rock berms, buttresses, or shear trenches incorporated into reconstructed fills. For example, a rock buttress was incorporated into a reconstructed shale embankment located along I-75, Tennessee. A system of horizontal drains supplemented fill reconstruction, as shown in Figure 47. Soil and shale materials will be most common in slope reconstruction. These materials

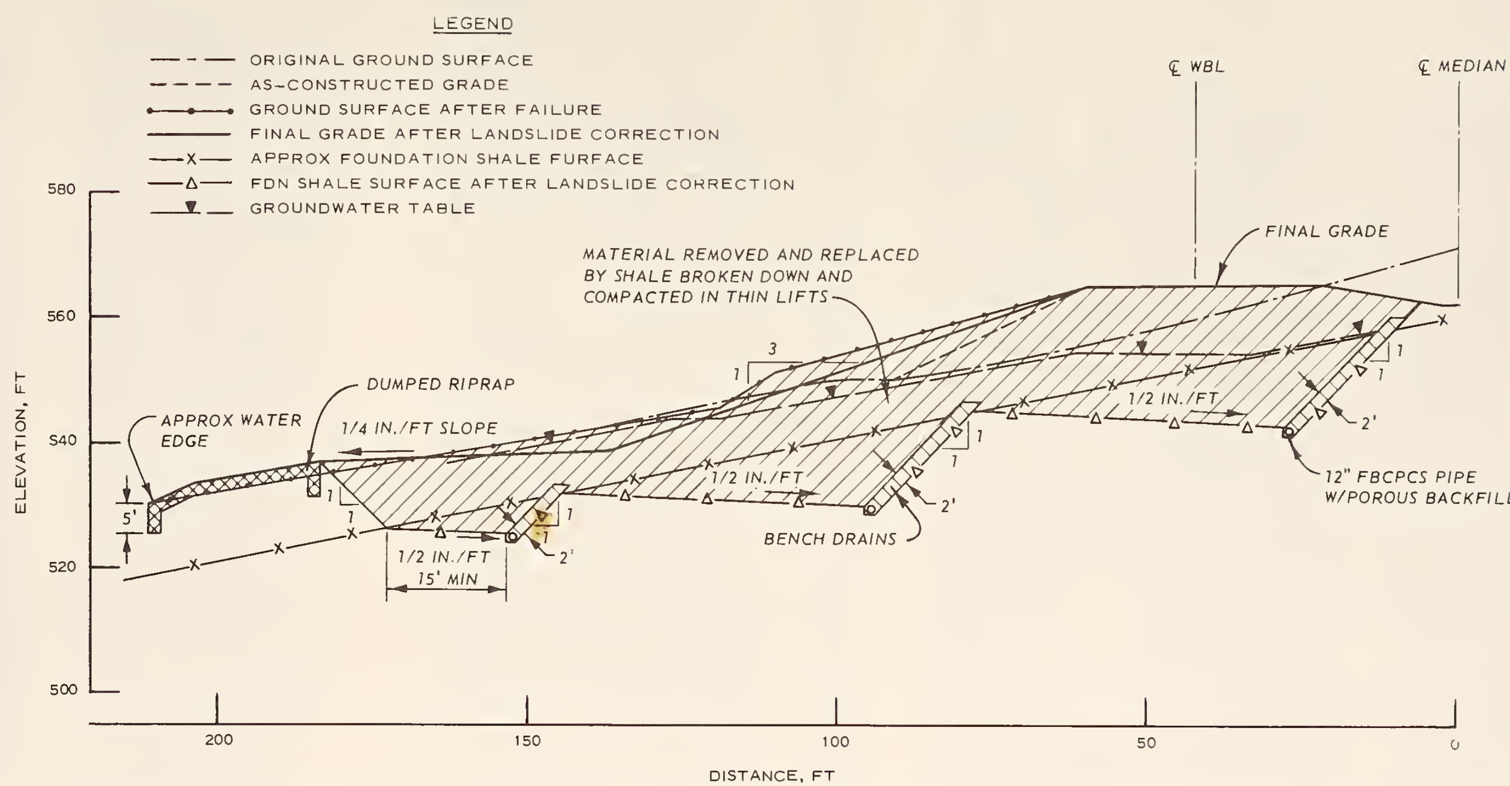

Figure 64. Remedial treatment of a shale embankment slide incorporating bench drains along I-74, Indiana (courtesy of the Indiana State Highway Commission)

should be compacted during placement to insure adequate density and shear strength. Benching, moisture control, thin lifts, and special procedures to break down shale materials should be used to obtain adequate compaction. The reconstructed fill (shown in Figure 64) consisted of shale materials broken down and compacted in thin lifts.

289. A critical factor in slope reconstruction is maintaining the stability of the remaining embankment slope after excavation. Stability analyses should be conducted to determine safe excavation slope angles and depths. Groundwater conditions should be considered to determine whether subsurface drainage is required to insure stability during the excavation and reconstruction period. If necessary, pumped vertical wells should be installed behind the proposed excavation to lower the groundwater table during reconstruction. For example, pumped vertical wells (see Figure 52 and discussion, paragraph 260) were installed to maintain stability during reconstruction of a sidehill shale embankment on I-75 in Tennessee. A large portion of the shale fill was removed and replaced with sandstone placed and compacted in 1-ft lifts. To minimize groundwater-related problems, it is recommended that embankment reconstruction be conducted during a dry season of the year.

Development and Implementation of Remedial Treatment Plans

290. Individual shale embankments require remedial treatment methods designed to meet site specific needs. Development and implementation of effective and economical site specific remedial treatment plans involves three major tasks: (a) site evaluation, (b) preparation of remedial treatment alternatives, and (c) final selection and construction. Discussion and recommendations concerning procedures and considerations under each task are presented below. Application of geotechnical engineering principles combined with engineering experience, judgment, and ingenuity are recommended in development and implementation of remedial treatment plans.

Site evaluation

291. Site evaluation is the initial phase in developing material treatment plans. An adequate site evaluation should be conducted to determine the extent and cause of embankment distress, remedial treatment needs, and basic parameters necessary to prepare effective remedial treatment plans. Site evaluation methods are presented in Part VIII. Essentially, site evaluation should include direct observation, measurement, or estimates of surface features and embankment and foundation characteristics and engineering properties. Information from site evaluations should include:

a. Embankment configuration, site geology, and surface topography.

b. Details and progress of present (and past) embankment distress. Amount and rate of embankment settlements; location, orientation, and width of surface cracks; location and height of failure scarps; depth and configuration of existing (or potential) shear surfaces and location of toe heave, if visible.

c. Embankment and foundation profile and material properties. Zonation or layering of embankment materials; shear strength, compressibility, grain-size distribution, and porosity of embankment materials and foundation soils; foundation rock stratification, including orientation and shear strength of bedding and joint surfaces along which slippage has or could occur.

d. Surface drainage conditions. Location and description of blocked or damaged drains and when blockage or damage occurred (i.e., before or after the time slope movement occurred); location and extent of cracks and/or depressions in the pavement, median, or embankment slopes in which water can accumulate and infiltrate the embankment.

e. Groundwater conditions and seepage parameters. Existing (or potential) groundwater table elevations and porewater pressures within the embankment and foundation; saturated zones or perched water tables within the embankment; seepage exits on the embankment slopes; foundation rock stratification and location of permeable water bearing layers intercepting sidehill and transitional fills; permeability of embankment and foundation materials.

The completeness and accuracy of site evaluation investigations will vary depending on site conditions and previous experience. Additional site investigations should be conducted, as necessary, when preparing remedial treatment alternatives, as discussed subsequently.

Preparation of remedial treatment of alternatives

292. Technically feasible alternatives should be prepared to meet remedial treatment needs as determined from site evaluation investigations. Permanent and temporary support requirements should be considered in preparing remedial treatment plans. Three or four alternatives should be prepared for providing permanent remedial treatment. Temporary (or emergency) treatment requires preparation of only one or two alternative remedial treatment plans.

293. The primary consideration in remedial treatment of shale embankments should be surface and subsurface drainage methods. Drainage methods are a recommended integral part of most remedial treatment plans.

All remedial treatment plans should include surface treatment and drains designed to minimize infiltration of surface water. Subsurface drainage is essential in treatment of sidehill and transitional fills. Certain types of subsurface drains can be rapidly installed (i.e., horizontal drains, pumped vertical wells) and are recommended when temporary (or emergency) support is required. Early implementation of subsurface drains should be conducted, when feasible, to halt embankment distress before extensive failure can develop.

294. Remedial treatment in addition to drainage methods will often be necessary when significant improvement in slope stability is required. Primary consideration should be given to adding fill for flattening slopes and constructing berms. Retaining walls for supporting slope-flattening or berm fills should be considered where right-of-way and/or suitable borrow materials are limited. A special type of retaining method (a row or rows of closely spaced piles) can be rapidly installed and should be considered when temporary (or emergency) support is required. Where embankment distress is caused largely by foundation shear failure, foundation shear trenches should be considered to supplement slope-flattening or berm fills. Embankment reconstruction involving combinations of material replacement, flatter slopes and berms, and shear trenches should be considered where large settlements, shear displacements, and/or shale degradation have severely weakened embankment and/or foundation materials.

295. Specialized stabilization methods including cement grouting and other cement, lime, and chemical treatments should be considered under certain conditions. Cement grouting should be considered when embankment settlements have been attributed to a high percentage of interconnected voids. Other cement, lime, or chemical treatments should be considered only on a trial basis at selected sites where risk of failure is minimal and substantial savings over more conventional remedial treatment methods can be realized. Expert guidance is required in the design and application of these methods.

296. Design of remedial treatment alternatives should be based on sound geotechnical engineering principles combined with engineering experience, judgment, and ingenuity. Design investigations should include a review of site evaluation data, past experience, and theoretical stability analyses. Stability analyses are an essential feature in design of economical and effective remedial treatment plans. Stability analyses aid in determining the significance and interaction of design variables and provide a quantitative basis for designing remedial treatment methods consistent with engineering judgment and experience. Stability analyses should be conducted in design of permanent or temporary support (including temporary stability of slopes excavated in construction of permanent remedial treatment). Acceptable factors of safety can vary, depending on the accuracy and confidence in design parameters and the risk of failure. It is recommended that factors of safety for permanent remedial treatment range from 1.25 to 1.5 and from 1.1 to 1.3 for temporary support.

297. Stability analyses methods are presented in the discussion of embankment design (Part IV). Circular arc and/or wedge stability analyses should be conducted. Circular arc analyses are generally applicable to shear failure within fill materials and soil foundations (see for example, Figure 49). Wedge analyses are generally applicable to shear failure along weak planar bedding contacts within soft foundation shales (see for example, Figure 57). Groundwater conditions and pore-water pressures should be considered in stability analyses. Shear strengths of existing shale fill and foundation material should be determined from laboratory tests, field tests, back analyses of distressed or failed slopes, or estimates based on previous experience or testing (see paragraphs 218 to 230). Laboratory tests or estimates are also required to determine shear strengths of (a) replacement fill used in slope reconstruction, (b) fill added to flatten slopes and/or construct berms, and (c) fill placed as shear trench or retaining wall backfill.

Final selection and construction

298. Final selection from among the remedial treatment alternatives should be based primarily on construction costs; however, several other factors should also be considered in making a final decision. Engineering judgment and experience should be applied in considering the following factors:

a. Construction costs.

b. Construction experience and time frame.

c. Potential construction problems.

d. Postconstruction maintenance and monitoring.

e. Accuracy and confidence in design parameters.

f. Risk of failure during or after construction.

g. Advantages and limitations of each alternative.

h. Environmental impact.

Construction should be conducted according to detailed plans and specifications with field supervision and inspection from qualified personnel. A construction quality control program should be conducted by contractor and/or State personnel. Geotechnical expertise should be readily available to the project engineer for advice and questions concerning treatment procedures. Significant construction modifications should be reviewed for acceptance or suitability by persons responsible for design of the remedial treatment plan.

299. A plan of postconstruction monitoring and maintenance should

be implemented following construction. The following items should be observed and additional remedial treatment or maintenance applied as necessary:

a. Embankment settlement and lateral deformation.

b. Pavement distress and surface cracking.

c. Erosion and cracking of embankment slopes.

d. Function and discharge from surface and subsurface drains.

e. Groundwater table elevation and embankment and foundation pore-water pressures.

The intensity of site monitoring will depend on site conditions, risk of failure, and critical factors in the remedial treatment plan design. It is particularly important to check and maintain the operation and effectiveness of subsurface drainage installations to insure that groundwater levels and pore-water pressures do not exceed safe values as determined from theoretical stability analyses. Site monitoring and maintenance plans should be modified periodically based on accumulated data.

APPENDIX A: SHALE TEST PAD CONSTRUCTION AND EVALUATION

1. This appendix contains recommended procedures and an example* for shale test pad construction and evaluation (used with permission of the Indiana State Highway Commission). The main purpose of test pads is to determine the required procedures (watering, disking, and number of compactor coverages) for soillike shales to obtain the desired percent compaction (related to percent maximum T-99 dry density) dictated by allowable long-term settlement and stability criteria. Test pads may be necessary at different locations, depending on variations in shale materials from various cuts or borrow sources.

Test Pad Configuration

2. The number of test pads at any one location depends on the compactors specified by procedural provisions. For one type of compactor, one test pad may be enough to develop required processing and number of coverages. (One coverage is one pass of the compactor over the entire area being compacted.) However, it is usually necessary to use two different types of compactors, such as a heavy (30-ton) tamping compactor and a heavy, vibratory pad drum compactor or a 50-ton, four-tired pneumatic roller. In the case of two compactor types, three test pads are required to allow various combinations of the number of coverages by each compactor to establish the optimum number of coverages (as listed in Table 18). A well-compacted, firm base that will not settle significantly is required for the test pad areas.

3. An example of a plan view for three test pads is shown in Figure 65. Laths with colored flagging were set along the edge of the pads as reference markers and are used to find randomly selected test locations. As shown in Figure 65, 56 possible test locations are defined. Numbers are selected at random to define test locations for gradation samples of the processed loose lift prior to compaction and a minimum of five test locations for in-place density and water content tests after each compactor coverage. The random selection of remaining test locations is repeated for each compaction coverage. This procedure provides a simple random sampling plan. Usually one test pad can be constructed, compacted, and tested in one day.

Construction and Testing

4. Shale test pads should be a part of the regular construction

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* Sisiliano, W. J. et al., "Report of a Shale Test Pad, R-Contract No. 10783, TQF-Project No. 105-1(1) Const., S.R. 145 in Orange County," Indiana State Highway Commission Division of Materials and Tests, Soils Department, Indianapolis, IN, May 1978.

Table 18. Scheme for Two Compactor Types and Three Test Pads

Scheme I

Test Pad	Compactor	No. of Coverages
I	A	2
	B	6
II	A	3
	B	6
III	A	4
	B	6

Scheme II

Test Pad	Compactor	No. of Coverages
I	A	6
II	B	6
III	A	3
	B	6

Notes:

1. Compactor A is usually a heavy (30-ton) self-propelled dual drum tamping roller (small area square or pointed feet may be necessary to break down hard shales or limestone pieces mixed with softer shales).
2. Compactor B is usually a heavy self-propelled vibratory roller with pad drum or a 50-ton pneumatic 4-wheeled roller.
3. Heavier compactors may be necessary if 6 to 10 total coverages do not produce required compaction results.

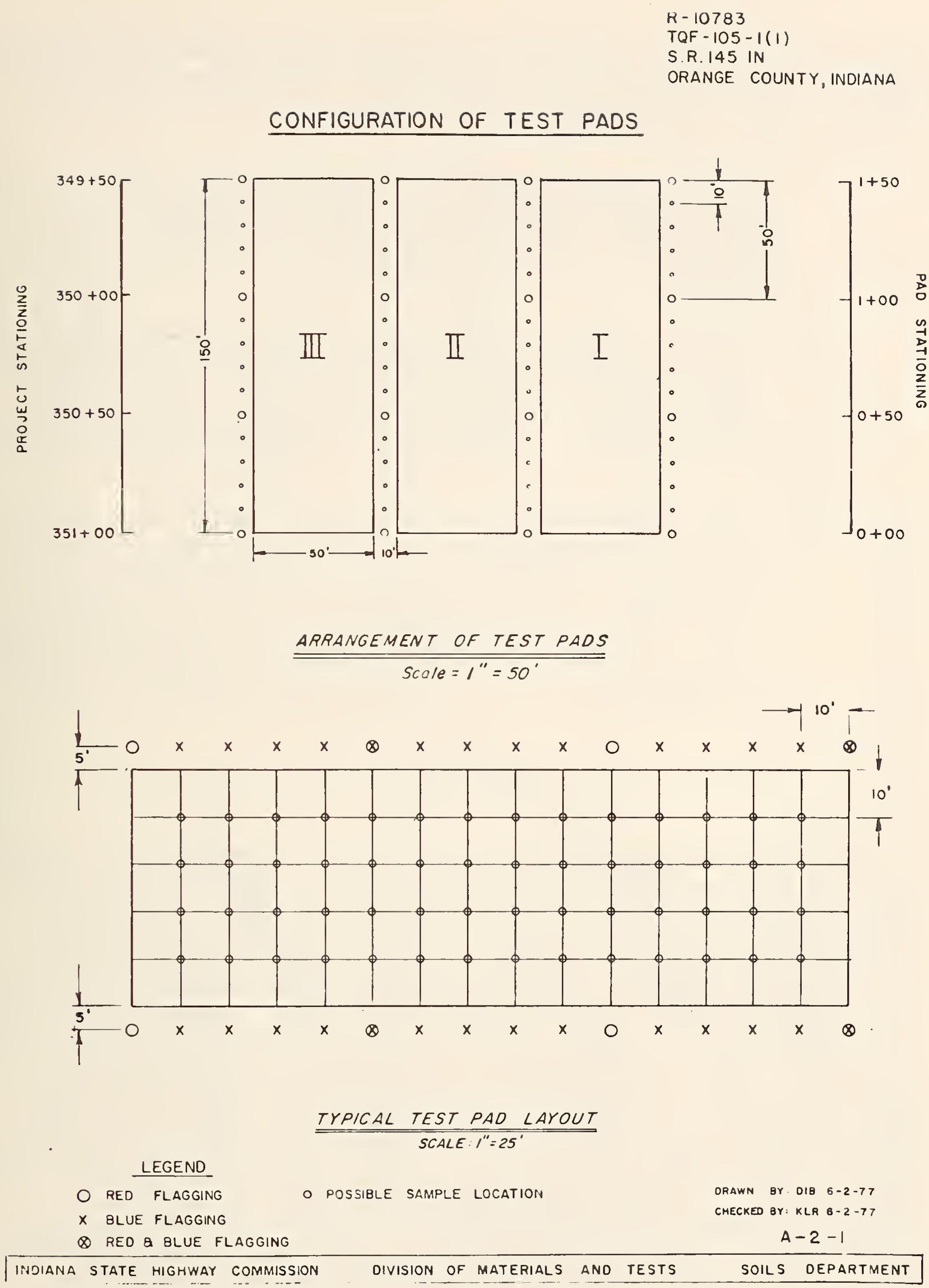


Figure 65. Example of test pad plan (courtesy Indiana State Highway Commission)

and serve as a training opportunity for the contractor and State Highway inspection and testing personnel. Realistic construction procedures and approved compaction equipment should be used. For each test pad, the following steps would be applicable:

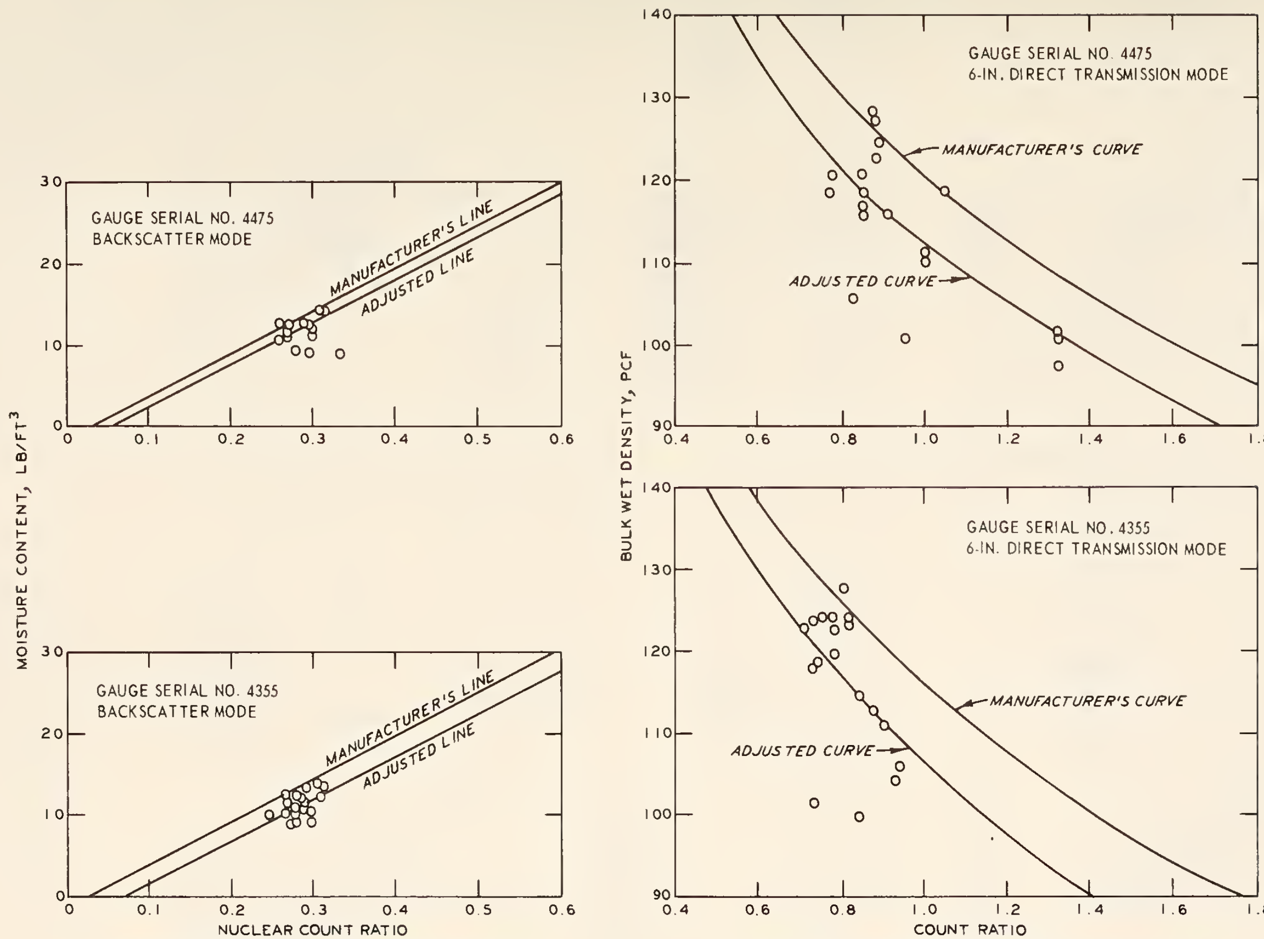
a. Place excavated shale material in 8-in. loose lift and spread using tracked dozer or compactor with dozer blade. Take gradation samples at five random locations. One sample for a laboratory compaction test should also be obtained.

b. Add water by spray bar and disk to bring shale material close to optimum water content (but not wetter than optimum). Random speedy moisture tests can be used to control addition of water. Several alternate disking coverages and small additions of water may be required to achieve maximum breakdown of shale and uniform distribution of moisture.

c. After each coverage by each compactor, tests at a minimum of five random locations should be performed using two test methods at each location. The nuclear moisture-density (backscatter mode for moisture and 6-in. direct transmission mode for wet density) method is the primary test. The 6-in. sand cone and speedy moisture tests should be used as a check. Samples for laboratory water content should also be obtained.

Evaluation

5. A major problem in evaluating the effect of increase in density with each additional coverage is the wide scatter of in-place moisture content and density. This problem is inherent in shale materials because of wide variations in gradation and density. A closely packed group of shale pieces, 1/4 to 3 in. in size, could give a high density and low moisture content, whereas an adjacent loose mixture of wet fines could give a low density and high moisture content. A high density also could be obtained if a large piece of shale or limestone happened to lie between the 6-in. direct transmission probe and the sensing element at the lift surface. To minimize this problem, nuclear moisture-density gauges must be properly calibrated and sometimes adjusted to the shale being tested as shown in Figure 66. In this example, the adjustments were necessary during initial testing on the basis of reliable sand cone tests and speedy moisture measurements. The latter proved to be as accurate as laboratory water content test results. An example of the scatter of test results obtained even in a carefully conducted test pad study is shown in Figure 67. From these and other test pad results, the following was concluded by the Indiana State Highway Commission Engineers:



a. MOISTURE CONTENT CALIBRATIONS ADJUSTED BY COMPARISON TO SPEEDY MOISTURE DATA, TEST PAD 1

b. DENSITY CALIBRATION CURVES ADJUSTED BY COMPARISON TO SAND CONE DATA, TEST PAD 1

Figure 66. Calibration adjustment for nuclear moisture density gauges (courtesy Indiana State Highway Commission)

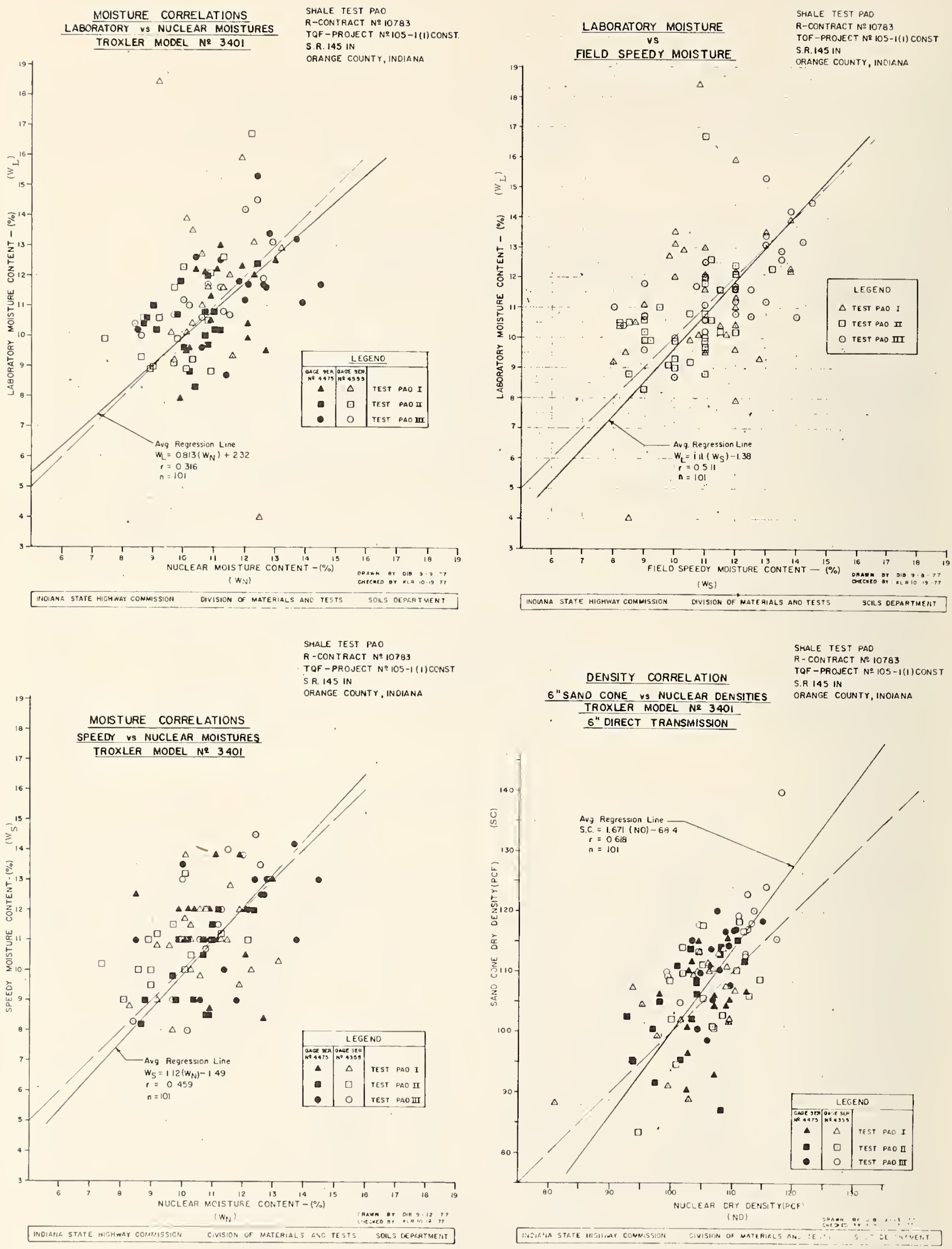


Figure 67. Example of inherent scatter in moisture and density data for a carefully conducted shale test pad program (courtesy Indiana State Highway Commission)

a. Test results obtained from the shale test pads indicate considerable variability in both the density and the moisture of the shale material after compaction. Until additional research, laboratory testing, and field testing on shale materials can be done, it is suggested that a procedural specification continue to be used to control embankment construction rather than moisture-density specifications.

b. The nuclear gauges produced results comparable to the speedy method for determining the in-place moisture content. It is therefore suggested that these nuclear procedures be used in conjunction with nuclear direct transmission for determining densities.

c. When the sand cone method is used, speedy moisture tests appear to be the optimum method to be used.

d. No evidence of any increase in accuracy could be found with the use of either the 6-in. sand cone or the nuclear gauges for determining compacted shale densities. When time and manpower requirements are considered, it is felt that a properly calibrated nuclear gauge used in the direct transmission mode is the superior method for determining in-place densities.

e. It would appear that the chemical differences between soil and shale is such as not to permit adjustment of the calibration curve by a limited number of comparison tests. It is therefore suggested that a study be performed, possibly by the Research and Training Center, to develop nuclear gauge calibration curves for different Indiana shales as they are encountered on construction.

6. The average dry density for each compactor coverage plotted versus the number of passes, as shown in Figure 68, can be used to select the optimum compaction procedure. In this case, three coverages by the static roller, followed by two coverages of the vibratory roller were needed to obtain 95 percent of maximum density. The use of test pads during construction provide a sound basis for selecting or modifying compaction procedures.

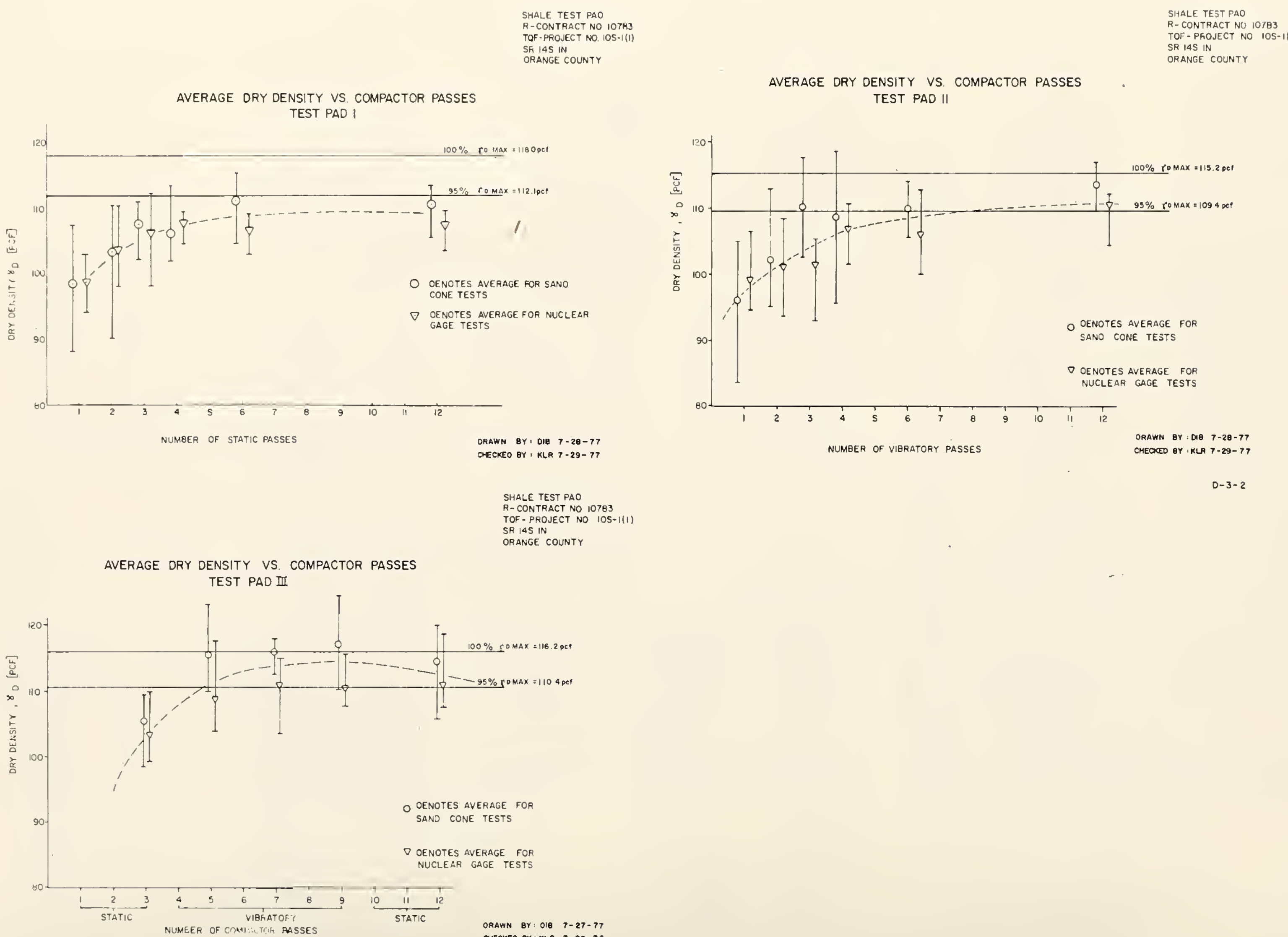


Figure 68. Example of evaluation of increase in density with increase in number of compactor passes (courtesy Indiana State Highway Commission)

APPENDIX B: PRESSUREMETER TESTING

Scope

1. This appendix describes the use of pressuremeter equipment for shale embankments. Recommended drilling techniques and test procedures are outlined. Techniques for data interpretation to estimate in situ horizontal pressure and modulus and shear strength have been described in Vol. 4 and by Baguelin, et al.*

Equipment and Operation

2. Pressuremeters consist of a radially expanding cylindrical probe connected by coaxial tubing to a pressure-volume control and monitoring system. The probe is lowered into a borehole to the desired testing depth, while the control and monitoring unit remains at the ground surface. The surface unit is used to apply pressure increments to the probe and to measure the corresponding volume increase. The resulting pressure-volume relationship (corrected for probe stiffness) is analogous to a stress-deformation curve.

Types

3. Figure 69 shows a diagram of the Menard (1975)** equipment with an NX long probe. This equipment (Models G and GA for soils) with various probe sizes applicable to shale embankments as listed in Table 19 is available through the U. S. licensee (Menard, Inc., 10 Duff Road, Pittsburgh, PA 15235, telephone number 412-243-0600 and Mounted Route 522B, Chestertown, IN 46304). Testing services are available from Menard Pressuremeter Testing Services, P. O. Box 787, Idaho Springs, CO 80452; telephone number, 303-567-4018. The Model G has a volume capacity of 1600 cm^3, while the model GA has a capacity of 800 cm^3, which may not be adequate for softer shale embankment materials.

4. Two self-boring pressuremeters that minimize soil disturbance have also been developed; the French PAFSOR (for stiff soils)† (not

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* Baguelin, F., Jezequel, J. F. and Shields, D. H., The Pressuremeter and Foundation Engineering, Trans Tech Publications, Rockport, Mass., 1978.

** Menard, L., "The Interpretation of Pressuremeter Test Results," Sols, Soils, No. 26, 1975.

† Baguelin, F. and Jezequel, J. F., "Further Insights on the Self-Boring Technique Developed in France," Proceedings of the Conference on In Situ Measurement of Soil Properties, American Society of Civil Engineers, Vol. 2, 1976.

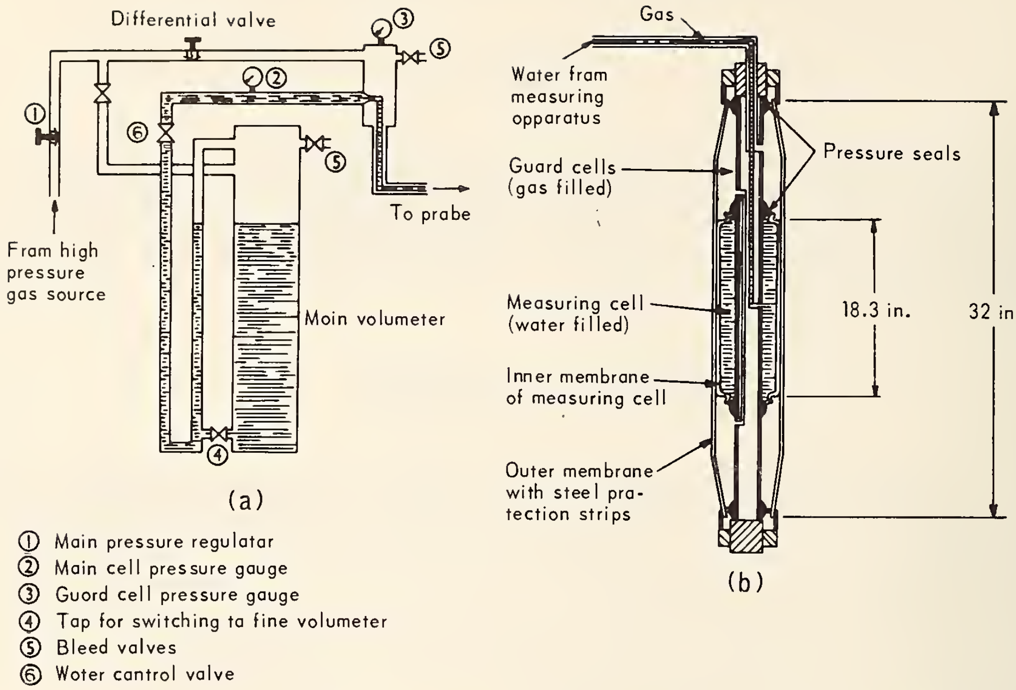

Figure 69. Menard pressuremeter (Type G): (a) volumeter, pressure circuits; (b) partially expanded probe (NX long) (after Marsland and Randolph, 1977)

Table 19. Pressuremeter Probe Dimensions and Borehole Diameters

DCMA Code	Outer Diameter of Probe* in.(mm)	Initial Volume of Probe cm^3	Length of Measuring Cell, in.(mm)	Borehole Diameter in.(mm)	
				Minimum	Maximum
BX	2.28(58)	517	8.27(210)	2.36(60)	2.60(66)
NX(short)	2.91(74)	808	8.27(210)	2.99(76)	3.15(80)
NX(long)	2.91(74)	1688	18.31(465)	2.99(76)	3.15(80)

* Varies depending on type and condition of outer sheath; values given are for rubber sheath with filled probe under 0.5 kg/cm^2 water pressure and no gas pressure. For a urethane sheath, the outer diameter is 2.85 in. (72 mm) for filled probe under 1 kg/cm^2 pressure and no gas pressure.

available in the U. S.) and the British Camkometer* (available through the U. S. licensee, Terrametrics, 16027 West 5th Avenue, Golden, CO 80401; telephone number, 303-279-7813). The Camkometer includes a gas pressure transducer, three feeler gauges, and an exterior pore-water pressure cell in the probe that give direct measurements of the probe pressure, radial expansion, and in situ pore water pressure. The self-boring pressuremeters (being evaluated by California Transportation Laboratory, CALTRANS) are limited to soils with small gravel sizes and are not appropriate for shale embankments except when used in a predrilled hole. Use of the self-boring equipment is not discussed in this appendix since experience in shale embankments has included only the Menard-type pressuremeters.

Operation

5. The Menard pressuremeter probe consists of an inner measuring cell filled with water and an outer membrane or sheath that forms guard cells filled by pressurized air or CO_2 above and below the measuring cell (Figure 69). The guard cells exert pressure on the ends of the measuring cell and restrict the measuring cell to a right circular cylinder during expansion. This restriction causes a uniform pressure on the borehole wall (plane-strain condition) over the length of the measuring cell. The differential valve (Figure 69a) reduces the pressure of the guard cells (1.0 kg/cm^2 to 2.0 kg/cm^2) below that of the measuring cell to ensure that the measuring cell pushes against the outer membrane (or sheath) initially and as it expands during the test. A net difference in pressure of at least 1.0 kg/cm^2 is required between the measuring cell and guard cells. In an open hole, the head of water acting on the measuring cell increases to 1.0 kg/cm^2 at a depth of 33 ft. Above this depth and at all depths in fluid-filled holes, the differential pressure valve (Figure 69) is used to decrease the guard cell pressure.

6. Because the measuring cell expands against the outer sheath, the probe must be calibrated for the sheath stiffness (inertia test). Calibration of the volume expansion of tubing and compression of the outer sheath under test pressures is not significant for the large volumes (1000 to 1500 cm^3) and low pressures (5 to 20 kg/cm^2) used in shale embankment testing.

Applicability for Shale Embankments

7. The Menard pressuremeter with a BX-size probe has been used in Tennessee (Vol. 1 and 4) and Indiana (Vol. 4, page 29) and with the

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* Windle, D. and Wroth, C. P., "The Use of a Self-Boring Pressuremeter to Determine the Undrained Properties of Clays," Ground Engineering, Sep 1977.

NX long probe during the Phase III studies (Vol. 4). Since shale embankments generally contain large pieces of intact shale, siltstone, sandstone, or limestone, the test section of the borehole should be as large as possible to overcome the size effects of the large pieces (6 to 12 in.). Even with the NX long probe, which has a measuring cell length of 18 in. (Table 19), excessively high pressures and/or nonuniform expansion of the probe can occur when the borehole test section includes one or two 6-in. or larger pieces of intact shale, siltstone, or harder rock. The BX probe with a measuring cell length of 8 in. would produce misleading results for test sections containing 3-in. or larger pieces of intact shale or rock. In spite of this problem, reasonable results can be obtained under the following conditions:

a. Performing tests in a borehole of the proper diameter, carefully made to minimize disturbance.

b. Testing within 15 to 20 minutes after advancing the borehole to the test depth to minimize borehole deformation.

c. Performing tests at frequent depth intervals (3 to 5 ft) to establish an upper and lower trend in test results and thus probable depths where large rock pieces are producing misleading results.

d. Correlating test depths with the drilling log to identify the depth of large rock pieces, rocky layers, fine-grained material layers, soft weathered shale, etc.

8. Since the pressuremeter test deforms the borehole wall horizontally, the estimated modulus and shear strength may be different than the modulus and shear strength determined from triaxial compression tests on undisturbed samples loaded in the vertical direction. The difference would be significant for shale embankments in which the stress-strain properties are significantly different in the horizontal and vertical direction. However, this aspect is minor when compared to variations caused by the heterogeneous nature of most shale embankments.

9. From a theoretical finite element study, Fragaszy and Cheney (1977)* concluded that the effects of borehole imperfections, soil inhomogeneity, and the length of the pressurized zone were small ($\pm$ 10 percent) under the following conditions:

a. The variation in radius is kept below 10 percent.

b. Many tests are performed to detect hard spots.

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* Fragaszy, R. J. and Cheney, J. A., "Influence on Borehole Imperfections on a Pressuremeter," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 103, No. GT9, Sep 1977.

c. A minimum length-to-radius ratio of 10 for the pressuremeter geometry is used.

The BX probe with a length-to-radius ratio of 7 (8.29/1.14) does not meet the above criterion, while the NX long probe with a length-to-radius ratio of 12 (18.3/1.46) exceeds the criterion.

Calibration and Test Procedures

Calibration

10. The stiffness of the outer sheath of the pressuremeter probe has a large influence on the test results. (Urethane sheaths are stiffer than metalic sheaths.) Calibration of the sheath stiffness is termed an inertia test and is performed at the ground surface with the probe (filled and checked for leaks) in an upright position with no confinement of the sheath. A pressure differential of at least 1 kg/cm^2 is necessary to ensure that the measuring cell pushes against the outer sheath. The volume expansion is measured for pressure increments of 0.5 to 1.0 kg/cm^2 up to a maximum pressure of 6 kg/cm^2 (to prevent the measuring cell or unrestrained sheath from bursting). The measured inertia pressure for a given volume expansion is subtracted from the borehole test pressure at that same volume. The stiffness of urethane sheaths changes with continued use (Vol. 4, Figure 40) and inertia calibration tests are required at the beginning and end of each day's testing.

Borehole preparation

11. Procedures for analyzing pressuremeter data require an accurate estimate of the in situ horizontal pressure (p_o) as a basic reference state. The disturbance, caused by drilling and subsequent wall deformation, has a significant effect on the accuracy of p_o estimates. Consequently, the borehole segment to be tested should be formed with the least possible disturbance and tested without delay. To minimize disturbance, it is recommended that the borehole be drilled and tested in 5- to 10-ft increments using the following procedure:

a. Advance the borehole to within 3 ft of each test depth using a bit and compressed air or a drive sampler having a diameter of 1/2 to 1 in. larger than the pressuremeter probe.

b. Drill the 5- to 10-ft test segment using a tricone roller bit and compressed air to form a hole with a diameter of not more than 1/8 to 1/4 in. larger than the deflated probe.

c. Perform the pressuremeter test(s) without delay.

12. Borings in most shale embankments will remain open but can ravel when a wet, gravelly layer is encountered. Compressed air is recommended rather than drilling fluid (mud with a unit weight of about 70 pcf) for the following reasons:

a. Loss of drilling mud into cracks and voids could increase the danger of a slide.

b. Drilling mud could cause softening of the shale and soil around the borehole.

Compressed air would not dry out the material appreciably during drilling of the 5- to 10-ft test segment.

Test procedure

13. After the probe has been filled, de-aired, and calibrated for stiffness of the outer sheath (inertia test), it is deflated. An appropriate differential pressure is set (be adjusting the differential valve, Figure 69), and the initial volume reading is recorded. The probe is then lowered (with the water control valve closed, Figure 69) to the desired test depth and allowed to expand (water valve opened) under the initial pressure from the head of water above the probe measuring cell. The volumeter is monitored and volume readings are recorded periodically, as indicated in Figure 70. When no further increase in probe volume occurs, the test is started.

14. The pressuremeter test is performed by applying pressure increments and recording volumes after 15, 30, and 60 seconds for each increment, as shown in Figure 70. For soft in situ materials, pressure increments of 0.5 kg/cm^2 are recommended without waiting the 3-minute standard time period after the first pressure increment is applied. Some trial and error is usually necessary to find the appropriate pressure increment magnitude. If too large an increment is used, volume expansion will reach the capacity of the volumeter (1600 cm^3 for the Model G equipment) before sufficient pressure increments are applied (minimum of 6 to 10 for a complete test). As the pressure increments are applied, the creep volume (60-second volume minus 30-second volume for each increment) and difference in volume at 60 seconds for successive increments are calculated and recorded (Figure 70). The test is stopped when the creep volume becomes high (increase of 10 cm^3 for an applied increment of 1 kg/cm^2). The creep volume or the 60-second difference can be used to judge when pressure increment magnitudes can be increased or need to be decreased (Figure 70) to better define the shape of the pressure-volume curve. Expansion of the probe should be limited to less than twice the initial volume (total volume expansion of 1600 cm^3 for the NX long probe) to prevent bursting of the measuring cell.

PRESSUREMETER TEST DATA

SITE: OH I-74 at S.R. 128, east side.	DATE: 4-9-76	BORING NO. PM2 Sta 424+80	DEPTH: 23 ft	OPERATOR: WES & GB	PAGE 1 OF 1

PROBE SIZE & TYPE	REMARKS
NX long with urethane sheath (4th test)	Boring air flush drilled to 32 ft on 4-8-76 (dry hole). Pressure differential = 0.5 kg/cm. Initial volumeter reading was 130 cm^3 with filled probe at surface. Hydrostatic pressure at 23 ft = 27.5 x 0.03 = 0.8 kg/cm^2.

TIME	LOAD KG/CM^2 (Applied)	VOLUME - CM^3 15 SEC	30 SEC	60 SEC	CREEP (30-60)	DIFF. (60 SEC)	REMARKS
9:49	0	173					Probe at 23 ft with water valve
9:55	0	172					open.
9:59	0	171					
10:03	0	171	--	171	--		Equilibrium at 0.8 kg/cm^2
10:04	2	302	--	342	--	171	hydrostatic pressure.
10:07	2	At	three min:	388	--	46	
--	3	441	445	463	18	75	
10:10	4	499	500	502	2	39	
10:12	6	548	549	550	1	48	
10:14	8	598	602	608	6	58	
10:16	10	671	682	700	18	92	
--	11	745?	755	775	20	75	
10:17	12	827	841	865	24	90	
10:20	13	920	938	972	34	107	End of test.
Note: Total pressure at 23-ft depth equals applied load plus 0.8 kg/cm^2.							

Figure 70. Pressuremeter test data record

Test results

15. The 60-second volume readings for each pressure increment are used to plot the pressure-volume curve. An ideal pressure-volume (P-V) curve and creep curve are shown in Figure 71 to illustrate the type of response. The initial steep portion of the P-V curve reflects expansion of the probe against the borehole wall. As the curve flattens out, the borehole wall is pushed back into its initial undisturbed condition of zero radial strain at the in situ horizontal earth pressure, p_o (at rest earth pressure). At this stage, the creep curve decreases to a minimum. The flat portion of the P-V curve is termed the pseudo-elastic phase and is used to calculate shear modulus. The creep curve remains at a constant minimum value. As the shear resistance of the in situ material is reached, the P-V curve steepens rapidly, as does the creep curve to indicate the plastic phase. Finally, the limit pressure (P_L) is approached at a large deformation. The inertia curve is also shown in Figure 71 and must be substracted from the P-V curve to obtain the corrected pressure-volume curve. Figure 72 shows curves for an actual test (data from Figure 70). The corrected P-V curve is used to estimate p_o , shear modulus, and shear strength. The estimated value of p_o is usually taken as the pressure at the beginning of the straight-line portion of the corrected P-V curve and corresponds to the pressure near the first low point of the creep curve. An excellent and easily understood review of pressuremeter theory and testing procedures is given by Marsland and Randolph in "Comparisons of the Results from Pressuremeter Tests and Large Scale In Situ Plate Tests in London Clay," Geotechnique, Vol. 27, No. 2, 1977. Examples of pressuremeter tests and detailed analytical procedures are given in Appendix A of Vol. 4.

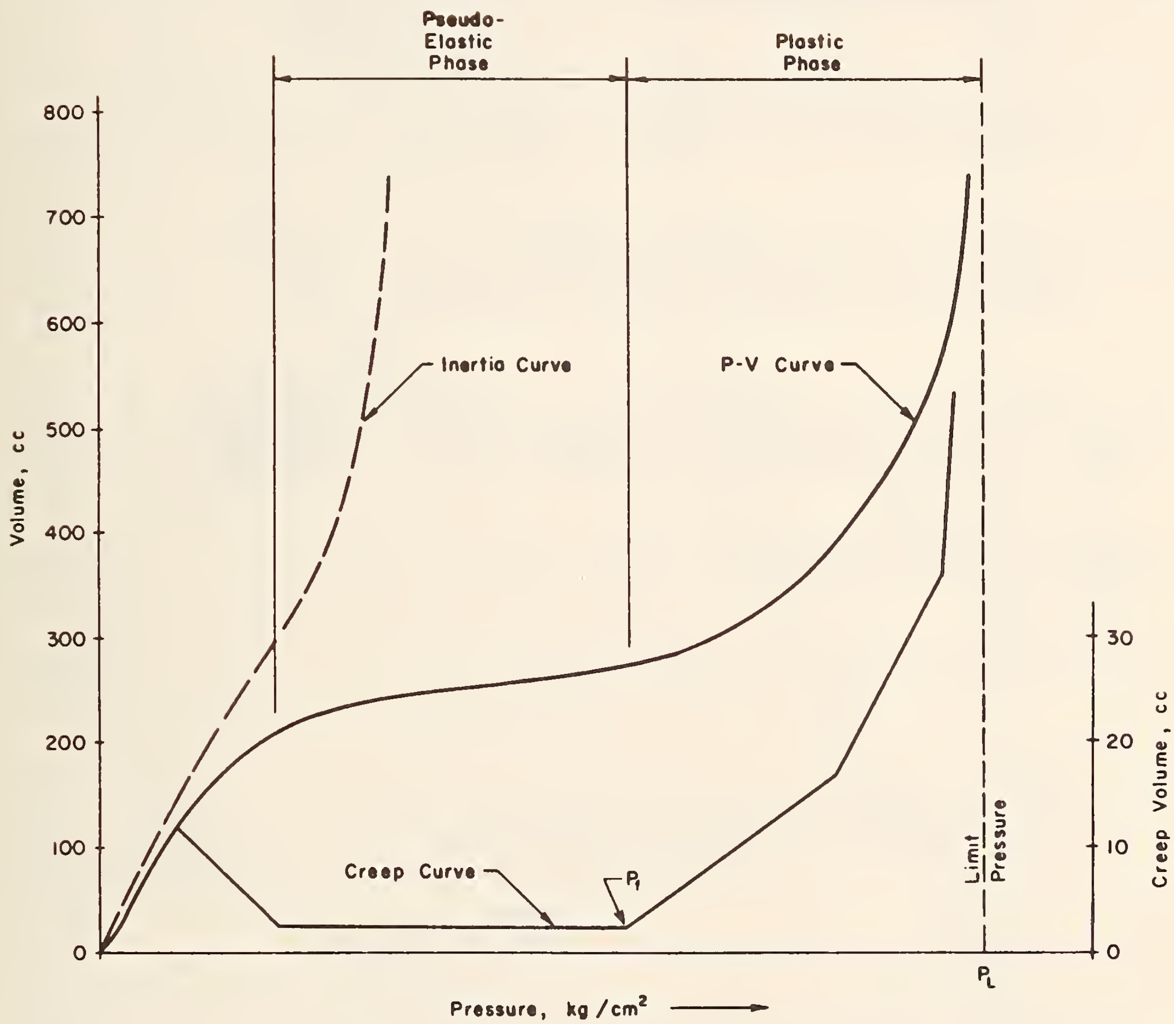

Figure 71. Curves derived from Menard pressuremeter data (Campbell and Hudson, 1969)

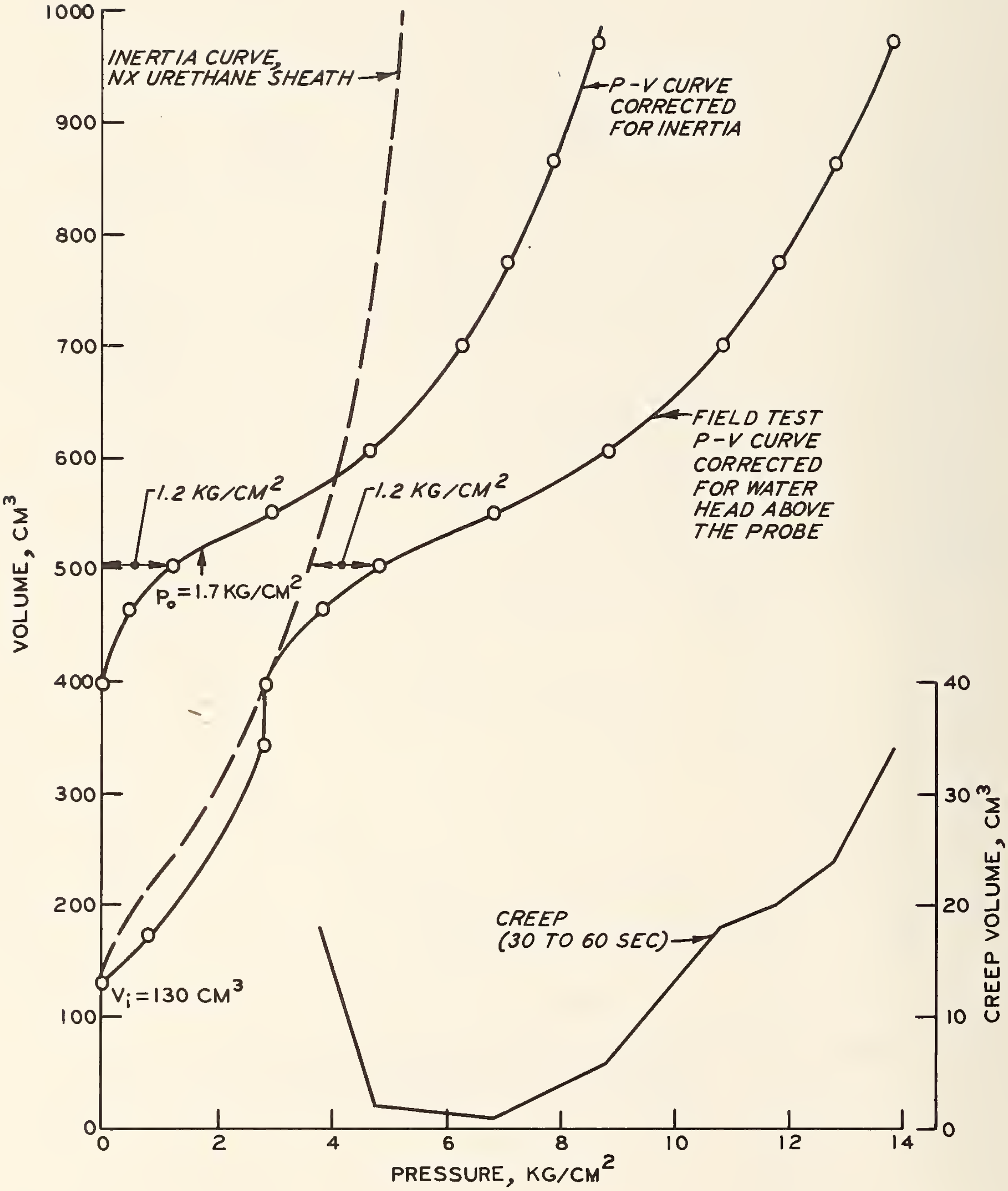

Figure 72. Example of pressuremeter curves for shale embankment test (data from Figure 70)

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

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* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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